NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP Report 406

Redundancy in Highway Bridge Superstructures

Transportation Research Board
National Research Council

REPORT DOCUMENTATION PAGE			Form Approved OMB No. 0704-0188
Public reporting burden for this collection of information is estimated to average 1 hour response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork reduction Project (0704-0188), Washington, DC 20503			
1. AGENCY USE ONLY (Leave blank)	2. REPORT DATE 1998	3. REPORT TYPE AND DATES COVERED Final Report	
4. TITLE AND SUBTITLE NCHRP Report 406: Redundancy in Highway Bridge Superstructures			5. FUNDING NUMBERS C12-36(2)
6. AUTHOR(S): Michel Ghosn and Fred Moses			
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) Department of Civil Engineering The City College The City University of New York New York, NY PB99-126799			8. PERFORMING ORGANIZATION REPORT NUMBER HR 12-36(2)
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) American Association of State Highway and Transportation Officials 444 North Capitol Street, N.W. Suite 249 Washington, D.C. 20001			10. SPONSORING/MONITORING AGENCY REPORT NUMBER
11. SUPPLEMENTARY NOTES Sponsored in cooperation with the Federal Highway Administration			
12a. DISTRIBUTION/AVAILABILITY STATEMENT Available from: Transportation Research Board 2101 Constitution Avenue, N.W., Washington, D.C. 20418			12b. DISTRIBUTION CODE: unlimited
13. ABSTRACT (Maximum 200 words) This report contains the findings of a study undertaken to develop a framework for considering redundancy in the design and in the load-capacity evaluation of highway bridge superstructures. A proposed specification to account for redundancy in design and evaluation is included. The contents of this report will be of immediate interest to bridge designers and bridge management engineers.			
14. SUBJECT TERMS Bridges, Other Structures, and Hydraulics and Hydrology			15. NUMBER OF PAGES 16. PRICE CODE
17. SECURITY CLASSIFICATION Unclassified	18. SECURITY CLASSIFICATION OF THIS PAGE Unclassified	19. SECURITY CLASSIFICATION OF ABSTRACT Unclassified	20. LIMITATION OF ABSTRACT

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Report 406

Redundancy in Highway Bridge Superstructures

MICHEL GHOSN and FRED MOSES
Department of Civil Engineering
The City College of
The City University of New York
New York, NY

Subject Areas

Bridges, Other Structures, and Hydraulics and Hydrology

Research Sponsored by the American Association of State Highway and Transportation Officials in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY PRESS
Washington, D.C. 1998

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

Note: The Transportation Research Board, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

NCHRP REPORT 406

Project D12-36(2) FY'93
ISSN 0077-5614
ISBN 0-309-06257-8
L. C. Catalog Card No. 98-60002

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

and can be ordered through the Internet at:

http://www.nas.edu/trb/index.html

Printed in the United States of America

FOREWORD

By Staff
Transportation Research Board

This report contains the findings of a study undertaken to develop a framework for considering redundancy in the design and in the load-capacity evaluation of highway bridge superstructures. A proposed specification to account for redundancy in design and evaluation is included. The contents of this report will be of immediate interest to bridge designers and bridge management engineers.

Both the AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *LRFD Bridge Design Specifications* require consideration of redundancy when designing highway bridges. These specifications provide limited guidance for determining when a structure is, or is not, redundant. The lack of guidance has led to a large variation in the interpretation of the specification.

This report is the culmination of NCHRP Project 12-36, which consisted of two phases. In Phase I, redundancy was defined and procedures were developed for quantifying redundancy in highway bridges. A direct calculation method and a simplified method using tabulated "system factors" were developed, and a framework for explicitly including measures of redundancy in design and evaluation was proposed. There was no report published on the Phase I effort.

In Phase II, the simplified method developed in Phase I was expanded to provide system factors for continuous span superstructures. The complete system factors tables are applicable to steel and prestressed concrete bridges comprised of I-beams or box girders. Although the system factors were developed using reliability analysis, they apply to bridges designed by AASHTO working stress, load factor, or load and resistance factor design criteria. These procedures for quantifying redundancy are significant because they are based on an assessment of the reliability of the bridge system rather than simply the individual bridge members. The introduction of system reliability may prove to be particularly advantageous for evaluating the load capacity of older bridges.

CONTENTS

AUTHOR ACKNOWLEDGMENTS

This report was written by Prof. Michel Ghosn and Prof. Fred Moses. The authors are grateful to Dr. John Kulicki for his comments and suggestions. The authors acknowledge the assistance provided by Mr. Linzhong Deng a research assistant at The City College of the City University of New York. Other students who worked on the project include Mr. Jian Ming Xu, Ms. Yi Liu, and Mr. Ge Chen, research assistants at The City College of New York, and Mr. Youhong Hang and Joe Oliver, research assistants at the University of Pittsburgh. The original version of the program NONBAN used extensively in this study was developed by Dr. Y. Liu, formerly a research associate at Case Western Reserve University.

REDUNDANCY IN HIGHWAY BRIDGE SUPERSTRUCTURES

SUMMARY

Bridge redundancy is the capability of a bridge superstructure to continue to carry loads after the damage or the failure of one of its members. Member failure can be either ductile or brittle. It can be caused by the application of large live loads or the sudden loss of one element due to fatigue, brittle fracture, or an accident such as collision of trucks, ships, or debris.

The structural components of a bridge do not behave independently but interact with other components to form one structural system. Current bridge specifications generally ignore this system effect and deal with individual components. Because redundancy is related to system behavior, this study attempts to bridge the gap between a component-by-component design and the system effect. This goal is achieved by introducing system factors that can be used to assess the member capacities of a bridge system as a function of its level of redundancy. The system factors are statistically based multipliers applied to the nominal member resistance and are related to the level of safety and redundancy of the complete bridge system.

The proposed system factors provide a method to ensure that highway bridges will provide a minimum level of system safety when the bridge is intact or after a component's failure. In this context, a bridge system is safe if: (a) It provides a reasonable safety level against first member failure. (b) It does not reach its ultimate system capacity under extreme loading conditions. (c) It does not produce large deformations under expected loading conditions. (d) It is able to carry some traffic loads after damage to a component.

The limit states that are checked to ensure adequate bridge redundancy and system safety are defined as

Member Failure. This is a traditional check of individual member safety using elastic analysis and nominal member capacity. The capacity of a bridge structure to support loads before the failure of any member is represented herein by a load factor, LF_1.

Ultimate Limit State. This is defined as the ultimate capacity of the intact bridge system. It corresponds to the formation of a collapse mechanism for bridges that have high levels of member ductilities. For less ductile bridges, the ultimate limit state is defined in terms of the loads that will render the bridge inadequate for use. For example, the bridge may reach its ultimate useful capacity when the concrete of a main mem-

ber reaches crushing. The ultimate capacity of a bridge structure to support loads is represented by a load factor, LF_u.

Functionality Limit State. This is defined as a maximum acceptable live load displacement in a main longitudinal member equal to the span length/100. A displacement equal to span length/100 is believed to be the maximum perceptible displacement that the public will accept. This displacement limit is proposed here on the basis of engineering judgment and is consistent with displacement levels used by other researchers and engineers. The capacity of a bridge structure at the functionality limit is represented by a load factor, LF_f.

Damaged Condition Limit State. This is defined as the ultimate capacity of the bridge system after damage to one main load-carrying element. The capacity of a damaged bridge structure to support loads before the ultimate capacity is reached is represented by a load factor, LF_d. The tables in this study assume damage scenarios that consist of the complete removal of a main girder for prestressed concrete and steel I-girder bridges. For spread boxes, the damage scenario assumes the loss in the capacity of one web and associated flange. For multibox beam systems, the damage scenario assumes the loss of one complete box. Other scenarios could be generated by the engineer when performing the direct analysis described below.

The load factors LF_1, LF_u, LF_f, and LF_d give the factors by which the weights of two side-by-side AASHTO HS-20 trucks are multiplied before the limit states are reached. Although other load models could be used, the HS-20 model is chosen here as a reference load because of its widespread use. Two side-by-side vehicles are considered to be the governing loading condition on the basis of the bridge loading models assembled during the calibration of the AASHTO LRFD bridge code.

To develop the system factor tables, numerous steel and concrete bridges were designed and analyzed. The load factors LF_1, LF_u, LF_f, and LF_d are calculated from an incremental structural analysis using a finite element package that considers the elastic and inelastic material behavior of the bridge members. The load factor values obtained from the analysis are compared with required values to verify whether acceptable behavior, nonfunctionality limits, or collapse states are observed under maximum expected truck loading conditions. In this study, the required load factors were determined from a system reliability calibration that examined load and resistance uncertainties as well as the performance of designs that are known to provide adequate redundancy. The system factors were developed to satisfy these required load factors. Thus, the proposed framework extends current design and evaluation methodologies, particularly the LRFD reliability calibration from the element level to the system level. The object of the proposed changes is to maintain an adequate level of system safety.

The proposed system factors are used as part of the design and evaluation equations for common-type bridges. The system factor tables developed in this study are applicable to prestressed concrete and steel bridges. I-beam bridges, as well as box-girder bridges with simple spans and continuous spans, are considered. The tables are given as matrices in function of the number of parallel girders and the girder spacings. Modifications based on bridge span lengths are also provided. The tables assume that all parallel girders of a bridge have equal capacities. Bridges with less than 40 deg skews are covered, although curved beams are not considered.

For bridges with configurations that are not covered by the tables, an engineer can check the redundancy of a bridge by performing a detailed incremental structural analysis. The proposed method is compatible with any acceptable AASHTO criteria including the WSD, LFD, and LRFD design codes as well as evaluation codes whether using inventory or operating rating levels. On the basis of the calibration performed in this study, a bridge will provide adequate levels of redundancy if all the following conditions are satisfied: (a) The system reserve ratio for the ultimate capacity, $R_u = LF_u/LF_1$, is greater than or equal to 1.30. (b) The system reserve ratio for the functionality limit, $R_f = LF_f/L_1$, is greater than or equal to 1.10. (c) The system reserve ratio for the damaged conditions, $R_d = Lf_d/LF_1$, is greater than or equal to 0.50.

Bridge designs that do not satisfy the criteria given above should be strengthened by requiring that their members be more conservatively designed than required by current standards. On the other hand, bridges that satisfy the above criteria should be rewarded by permitting less conservative member designs. This could be achieved through the application of system factors that are proportional to the level of compliance with the recommended criteria. The system factors obtained from the incremental analysis can be directly applied during the design process to strengthen nonredundant configurations or to reward redundant configurations. On the other hand, the system factors can be used to calculate rating factors for the evaluation of existing bridges. Thus, bridges that are not redundant must have their member capacities increased or, they will have lower ratings.

The framework proposed in this study is used to modify the ultimate capacities of main bridge members based on the superstructure's redundancy. Separate calculations should be performed to verify that the members' serviceability criteria, such as cracking and fatigue, are still satisfied. In addition, the capability of the connections to sustain the applied loads must be independently verified.

The proposed methods are calibrated by using reliability techniques. In this context, redundancy is defined as the difference between the reliability index (or safety index) of the bridge system and the reliability index of the members. The system factor tables and the load factors recommended for the direct incremental analysis were calibrated to ensure that highway bridges provide adequate levels of system reliability. Numerous bridge examples were evaluated in order to recommend target system reliability (safety) indices. Each bridge system's ultimate capacity as well as its functionality and capacity under damaged conditions was checked.

The calibration of the system factors and the recommended load factors was performed on the basis of the observation made in this study that a bridge will provide adequate levels of redundancy if all the following conditions are satisfied: (a) The difference between the system reliability index for the ultimate limit state and the reliability index for one member is greater than 0.85. (b) The difference between the system reliability index for the serviceability limit state and the reliability index for one member is greater than 0.25. (c) The difference between the system reliability index for the ultimate limit state of a damaged bridge and the reliability index for one member is greater than −2.70. A negative range for the system index compared with a member index in condition c reflects that a damaged bridge does not require the same reliability level as the original intact structure.

Although reliability techniques are used to develop the proposed methodology, the reliability model is transparent to the end user. To consider redundancy during the design and evaluation of a bridge structure, the bridge engineer can simply use the proposed system factors and the load factors from the incremental analysis without referring to reliability theory.

CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

Bridge redundancy, as normally defined, is the capability of a bridge to continue to carry loads after damage to or the failure of one or more of its members. Member failure can be either ductile or brittle. It can be caused by the application of large live loads, the sudden loss of one element due to brittle fracture, or an accident such as a collision by a truck, ship, or debris.

The capability of a bridge to continue to carry loads after a member's failure is due to its ability to redistribute these applied loads. This normally involves a potential for transverse and longitudinal redistribution. The transverse redistribution is usually a function of the longitudinal member properties, the deck slab, the effect of secondary members, the connections, and the geometric configuration of the bridge. Important geometric properties include the number of girders, girder spacing, number of spans, and the span lengths. The longitudinal redistribution is usually affected by the available ductility of bridge members especially at the continuous supports.

The 1996 AASHTO specifications (*1*) for the design of highway bridges recognize the importance of redundancy and require its consideration when designing steel bridge members. On the other hand, the specifications give limited guidance on how to define redundancy. The specifications state that a structure is "nonredundant when the failure of a single element could cause collapse." The specifications do not define collapse or the loadings associated with its occurrence. This omission has led to a large variation in interpretation and application in bridge design and analysis. In addition, the specifications do not require consideration of redundancy in the design of other than steel bridges.

The AASHTO LRFD (Load and Resistance Factor Design) specifications (Section 1.3.2) propose a format explaining how redundancy can be included in the design process by using "load factor modifiers" (*2*). As proposed, the load factor modifiers are a function of a subjective evaluation of the "operational importance" of a structure, the "level of ductility," and the "level of redundancy." Operational importance concerns "the consequences of the bridge being out of service" based on "social/survival and/or security/defense requirements." These criteria are similar to those used in the design for earthquakes. The classification of members as ductile or nonductile is left up to the designer. For concrete connections, the code mentions that ductility requirements are satisfied if the resistance of the connection is at least 1.3 times the maximum force effect of the adjacent components. The designation of a member as redundant or nonredundant is based on the "member's contribution to the bridge safety." Main elements whose failure is "expected to cause the collapse of the bridge are designated as failure-critical and the structural system is non-redundant." Collapse is defined as "a major change in the geometry of the bridge rendering it unfit for use." To include "importance, ductility and redundancy" in the design process, each one of these effects is assigned a factor of 0.95, 1.0, or 1.05. The total load modifier is the product of the individual factors. Values of the modifiers less than 1.0 are for bridges with high levels of ductility and redundancy and a low level of operational importance. Values greater than 1.0 indicate that bridge members must be strengthened because of potentially inadequate load redistribution capability or that the structure geometry must be changed. The definitions suggested in the LRFD recommendations (*2*) are subjective, and the proposed load modifiers need further refinement, although they are currently being implemented in practice.

1.2 SCOPE OF THE INVESTIGATION

The goal of this study is to develop a method that accounts for redundancy during the design and safety evaluation of highway bridges. This includes checking the redundancy of intact bridges under the effect of heavy loads as well as evaluating potential hazards to damaged bridges. Damage is defined as the removal of one main component of the bridge structure. In a variation of the format as proposed in the AASHTO LRFD specifications (*2*), the framework developed here consists of a set of system factors that can be included in bridge design and evaluation codes to account for the redundancy and ductility of bridge superstructures.

To analyze the redundancy of a bridge, the ideal situation would be to use a structural model and a finite element analysis package that consider the elastic and inelastic behavior of the bridge members. This program could evaluate intact bridges under the effect of heavy loads as well as consider different damage scenarios. The program could be used to check the structure to verify whether acceptable behavior, unserviceable conditions, or collapse states occur under maximum expected truck loading conditions. These programs, however, are not always available and more simplified approaches are needed.

Even when a program that can perform a nonlinear incremental analysis is available, there are still many safety-related decisions that must be made. These include: (1) the definition of the limit states or the safety criteria that should be considered; (2) the level of loads that must be carried before the limit states are reached; (3) the type of damage conditions that must be investigated; and (4) the inclusion of uncertainties in the analysis model. This study offers answers to all of these questions on the basis of available data on bridge behavior and load modeling.

Bridge redundancy is related to the bridge's geometry and its members' properties. Some bridges can be easily classified for both geometry and member properties. For example, bridges of the same material that have similar geometries and have members designed according to the same specifications can be expected to have similar levels of redundancy. These bridges will be referred to as "typical" bridges. In this report, typical bridges are studied as a group, and tables of system factors are developed to estimate and quantify their levels of redundancy.

Other bridges are unique because of their geometry and nontypical member properties; these have to be studied individually by the engineer performing the safety evaluation. This report provides general guidelines to help the engineer perform the redundancy evaluation of "nontypical" bridges by using a direct analysis approach.

The structural components of a bridge do not behave independently but interact with other components to form one structural system. Current bridge specifications ignore this system effect and deal with individual components. Because redundancy is connected to system behavior, this study will attempt to bridge the gap between a component-by-component design and the system effects. For typical bridges, this goal is achieved by introducing system factors that lead to an increase in the required member capacities of systems with low levels of redundancy. These system factors provide a method to control the redundancy of intact bridges and also consider the safety implications of a damaged bridge system.

The proposed system factors could be used in the routine design or strength evaluation of the components of typical bridges. The proposed system factors are suitable for inclusion as recommendations in the AASHTO bridge design and evaluation specifications using either WSD (Working Stress Design), LFD (Load Factor Design), or LRFD criteria. The tables in this study are applicable to simple-span and continuous prestressed concrete, I-girder bridges; simple-span and continuous multigirder steel bridges; simple-span prestressed concrete, spread box-girder bridges; and simple-span prestressed concrete, multibox beam bridges. The tables are given as a function of the number of parallel girders and the girder spacings. All parallel girders of one bridge are assumed to have equal capacities.

For bridges with nontypical configurations that are not covered by the tables, a direct analysis approach is recommended. This requires an incremental analysis to verify whether acceptable behavior, unserviceable conditions, or collapse states occur under maximum expected truck loading. General guidelines describing how to perform such an analysis are provided. These guidelines include the loads that should be applied, the limit states that should be checked for both intact and damaged conditions, and the target load factors that the bridge should sustain before these limit states are violated. Redundancy factors similar to the system factors developed for typical bridges can then be calculated from the results of the incremental analysis.

The system factors obtained from the tables and the redundancy factors obtained from the incremental analysis can be directly applied during the design process to penalize nonredundant configurations or to reward redundant configurations. On the other hand, these factors can be used to calculate rating factors for the evaluation of existing bridges. Thus, bridges that are not redundant must have their member capacities increased or, they will have lower ratings.

1.3 RESEARCH APPROACH

The framework proposed for implementing redundancy concepts in the design and evaluation of highway bridges will have two parts. The first part presents tables of system factors that would be used to modify component strengths based on the redundancy of typical bridge systems. The second part presents guidelines for the redundancy analysis and evaluation of any bridge system using a nonlinear structural analysis program. The latter option can be used to check any bridge system provided that an incremental nonlinear analysis program is available.

To achieve the goals of this study, the limit states that should be checked to ensure adequate bridge redundancy and safety are defined. These limit states ensure the safety of the complete bridge as well as its functionality. As defined in this study, functionality is meant to ensure that the bridge system does not sustain large levels of permanent deformations that would render the bridge nonfunctional after the passage of a combination of heavy trucks. In addition, limit state requirements for damaged conditions are also defined. Each limit state must satisfy a target system safety criterion. The target system safety criteria are calculated using structural reliability methods and bridge loading data and are based on experience with current bridge designs.

From the review of previous studies, the research team observed that bridge redundancy is a function of bridge type, bridge geometry, and member ductility. To study the interaction of these parameters and to develop the proper safety criteria, a nonlinear incremental structural analysis program was developed to perform the nonlinear analysis of common-type girder bridges. The program accounts for different types of material, member ductilities, and geometries. The validity of this computer program was tested by comparing its results with those of experimental full-scale bridge tests as well as published analytical studies. The program was developed for use during the calibration process. To use the direct analysis approach proposed in this study, the engineer could use any commercially available finite element package capable of considering material nonlinearity. To use the system factor tables, no additional structural analysis beyond that required in traditional design is necessary.

A large number of bridges were designed during the calibration process to cover all the typical geometries and properties of common steel and prestressed concrete bridges in the United States. The nonlinear analysis of these bridges was performed using the program developed in this study, first assuming that the bridges are intact and second assuming that one main load-carrying component was damaged. Damage is defined as the removal of one main load-carrying element from the structural model. Other damage scenarios could also be specified during the direct analysis at the discretion of the bridge owners.

For intact bridges, the conclusions obtained from the nonlinear analysis are presented as the load factors required to satisfy a functionality limit state as well as the load factors required for an ultimate capacity limit state. For damaged bridges, the load factors are presented only for the ultimate capacity. This presumes that damage will be detected during inspection, and thus no functionality check is required for damaged bridges.

To establish minimum redundancy requirements, reliability calculations were performed for all the typical bridges analyzed. The reliability calculations produce system safety indices for typical bridges on the basis of the results of the nonlinear analysis. The reliability calculations also provide member safety indices. Target system safety indices are extracted from these analyses by considering examples of bridges that are known to provide adequate redundancy. In addition, target relative safety indices are extracted to give the minimum reserve strength that should be provided by the system compared with the level provided by one member.

A bridge will have an adequate level of redundancy when the difference between the system reliability index and the member reliability index is higher than the target value. The target system index values are extracted based on reviewing numerous bridges that, according to current practice, are believed to provide redundant designs.

Using the target indices, a step-by-step procedure or a "direct analysis approach" is developed. The direct analysis approach can be used for any type of bridge system provided that a program capable of performing a nonlinear incremental analysis is available. General guidelines are calibrated to provide the appropriate loading conditions and load factors required to check the redundancy levels of bridge systems.

The direct analysis approach described is applicable to all AASHTO design and evaluation criteria, including WSD, LFD, and LRFD. The proposed approach is calibrated to provide a uniform level of relative safety indices. The relative safety indices are the differences between the system safety indices and the member safety indices.

Finally, using a reliability-based calibration, system factors are developed to reflect the redundancy of common-type steel and prestressed concrete bridges. These system factors are similar to the load modifiers proposed in the AASHTO LRFD specifications (*2*). The system factors can be used to modify the component strengths so that functionality and collapse limit states are satisfied with reasonably high safety margins for both intact and damaged conditions. The purpose of these system factors is to check the system design and redundancy of commonly used bridges without performing a nonlinear analysis.

1.4 REPORT OUTLINE

Chapter 2 of this report presents the methodology used in this project and the findings of this study. It reviews the basic concepts of bridge safety, defines the limit states used in this study, and discusses the different loading conditions and safety criteria considered in the analysis. These definitions lead to the direct analysis methods that perform the redundancy and system safety evaluation of bridge systems using an incremental structural analysis program. Finally, system factors for common-type steel and prestressed concrete bridges are presented. Chapter 2 also gives a set of specifications and commentaries that could be implemented in future editions of the AASHTO specifications.

Chapter 3 illustrates the practical application of the findings with detailed examples. Chapter 4 gives the conclusions and reviews the applicability and the limitations of the findings and suggests future implementations.

Technical details, calculations, and theories are given in the appendixes. Appendix A presents a review of current practice in the redundancy evaluation of structural systems and, particularly, bridges. Appendix B gives the results of the analysis of typical prestressed concrete I-beam bridges, including a sensitivity analysis to determine the important parameters that affect bridge redundancy and the reliability calculations. Appendix C performs the same tasks of Appendix B for steel I-beam bridges. Appendix D studies the redundancy and reliability of prestressed concrete spread box-girder and multibeam box bridges. The reliability calibration of the tables developed for the common-type bridges and the load factors used with the direct analysis approach is performed in Appendixes B, C, and D. Appendix E proposes a method to extend the results of this study to also include bridge substructures. Appendix F describes the program NONBAN that developed in this study to perform the analysis of the typical bridge configurations. It also provides the listing of program NONBAN as well as details about the program's logic and structure.

CHAPTER 2

METHODOLOGY AND FINDINGS

2.1 INTRODUCTION

Redundancy is the capability of a bridge superstructure to continue to carry load after the failure of one of its members. This means that the superstructure has additional reserve strength such that the failure of one member does not result in the failure of the complete superstructure. Member failure can be either brittle or ductile. It can be caused by the application of large live loads or the sudden failure of one element due to fatigue, brittle fracture, or an accident such as a collision by a truck, ship, or debris.

A convenient method to evaluate the redundancy of bridge systems would consist of developing a set of system factors that can be included as specifications in bridge design and evaluation codes. On the other hand, a direct analysis approach would consist of using a structural model and a finite element analysis program that consider the elastic and inelastic behavior of the bridge system. This program could evaluate intact bridges as well as the consequences of assumed damage conditions. The program could check the structure to verify whether its behavior is acceptable, whether it remains functional after the passage of expected heavy loads, or whether it would collapse under the effect of applied vehicular loads.

Many safety-related decisions must be made in order to develop the system factors or to use the direct analysis approach. These include (a) the limit states that should be checked; (b) the level of loads that must be carried by the structure before the limit states are reached; (c) the type of damage conditions that must be borne by the structure; and (d) the inclusion of uncertainties in the analysis model.

This chapter uses a simple system reliability model to provide answers to these important questions. The goal is to develop a set of system factors that can be included in bridge specifications and to provide guidelines describing how to evaluate the redundancy of bridge superstructures by using a structural analysis program.

Section 2.2 gives a brief discussion on the issue of bridge safety. Section 2.3 defines the limit states that are used in this study. Section 2.4 defines the load levels that are used in the analysis. Section 2.5 develops the reliability model used to establish the safety levels that should be satisfied. Section 2.6 determines the target reliability indices used to calibrate the proposed system factors as well as to develop the guidelines for the direct analysis approach. Section 2.7 proposes a direct analysis method that can be used in conjunction with any member checking criteria including WSD, LFD, and LRFD specifications. Section 2.8 develops tables of system factors that can be used for common-type bridge systems. These system factors can be used without performing additional structural analyses beyond those required for traditional design. Section 2.9 explains how the framework developed in this study can be used for the rating of existing bridges. Section 2.10 proposes a set of specifications that can be included in the LRFD code to account for bridge redundancy during the bridge design and evaluation process. Section 2.11 summarizes the findings of this chapter.

This chapter should be read in conjunction with Appendixes A through F. The appendixes review recent developments in subjects related to bridge redundancy, explain the theoretical background, describe the analytical and numerical models used in this study, give details of the program developed to perform the calculations, and analyze the results.

2.2 BRIDGE SAFETY

Current design and evaluation techniques deal with individual members and use procedures that ignore the effect of the complete structural system. As currently performed, the safety check verifies that the strength of each member is greater than the applied forces by a "comfortable" safety margin. The member forces are calculated by using an elastic analysis while member capacity (when appropriate) may be calculated by using inelastic member behavior. The safety margin is provided through the application of safety factors (load and/or resistance factors) that are calibrated on the basis of experience and engineering judgment (WSD and LFD) (*1*) or on a combination of experience and structural reliability methods (LRFD) (*2*). In addition to checking member safety, the traditional approach involves a check of member serviceability under working loads (*1*).

Although this traditional member-oriented approach has been used successfully for years, it does not provide an adequate representation of the safety of the complete bridge system. In many instances, the failure of an individual member does not lead to the failure of the complete bridge system. On the other hand, because of possible large deformations, the bridge may be inadequate for truck traffic at loads that are lower than those that will cause a system collapse.

Loss of member capacity is also a concern. Bridge members are often subjected to fatigue stresses that may lead to the fracture and loss of the load-carrying capacity of a main member. In addition, corrosion, fire, or an accident, such as a collision by a truck, ship, or debris, could cause the loss of a bridge member or the severing of the prestressing strands. To ensure the safety of the public, bridges should be able to sustain these damages and still operate, although at reduced capacity. Therefore, in addition to verifying the safety of the intact structure, the evaluation of a bridge's safety and redundancy should consider the consequences of the failure of a critical bridge member.

In summary, a bridge is safe if it (a) provides a reasonable safety level against first member failure; (b) provides an adequate level of safety before it reaches its ultimate system capacity under extreme loading conditions; (c) does not produce large deformations under expected heavy traffic loads; and (d) is able to carry some traffic loads after damage to or the loss of a component.

The concept of redundancy is related to the overall system capacity. This, in turn, is dependent on the bridge members' strengths. The issue of member serviceability is outside the scope of this study. However, member serviceability criteria, such as cracking and fatigue, should always be independently checked.

2.3 DEFINITION OF LIMIT STATES

To analyze the safety of bridge systems, their inherent capacities to carry loads before critical limit states are reached should be calculated. The system factor tables provided in Section 2.8 are calibrated based on the limit states discussed in this section. However, for engineering applications, redundancy can be included during the design and evaluation of typical bridge configurations by using the tables provided in Section 2.8, without performing the analyses described here.

On the basis of the concepts of bridge safety discussed in Section 2.2, this study investigates four critical limit states. The capacity of a bridge superstructure to carry live loads before these limit states are reached is related to the live load margin (R − D), where R is the bridge capacity and D is the dead load effect. To estimate the live load margin (R − D), an incremental structural analysis must be performed. This requires a structural model and a load configuration. For the sake of simplicity and consistency with current practice, the analyses performed in this study use two side-by-side AASHTO HS-20 trucks for loading. To estimate the live load margin for a given limit state, the HS-20 loads are incremented until the limit state is reached. Therefore, the bridge capacity represented by R − D is proportional to the factor that multiplies the weights of the two HS-20 trucks. This multiplier is referred to here as the load factor LF.

Although other load models could be used, the HS-20 trucks were chosen because of their widespread use. Two side-by-side vehicles are considered independent of the actual number of lanes on the bridge to obtain a common reference point for comparison. Two side-by-side trucks are used on the basis of the assumption that this configuration is the governing loading condition for simple-span bridges as determined by the bridge load studies performed for the AASHTO LRFD code (*2,3*). Section 2.7 discusses the generalization of the results obtained with the HS-20 load model to other load models as well as the extension of the results to bridges with other than two lanes of traffic.

The purpose of the load factor LF is to provide a convenient measure of the bridge capacity represented by R − D. This capacity depends on the position of the loads during the incremental analysis. For convenience and consistency with current practice, it is proposed to place the HS-20 truck loads in the position that will produce the highest bending stresses in a main member using a linear elastic analysis. Unfactored member strengths, unfactored dead loads, and two HS-20 trucks without impact factors are used in the incremental analysis. Because at this stage we are interested in studying the capacity of the structure, the dynamic factors are omitted from the analysis. The effect of the dynamic impact loads will be considered further when discussing the expected loads that will be applied on the bridge structure.

To be effectively used in the safety analysis of bridge systems, the limit states checked during the incremental analysis must provide "objective" and "universal" criteria. This means that the limit states should be easily defined with available structural analysis programs, and they should be applicable to all bridge types and configurations. Based on the discussion of Section 2.2, the following four limit states are chosen.

2.3.1 Member Failure

To be consistent with current evaluation techniques, it is proposed that the check of member failure be performed using an elastic analysis of the structural system. In the proposed procedure, member capacity is calculated using AASHTO member strength formulas without safety factors. As mentioned previously, the concept of redundancy is related to the bridge members' strengths; however, member serviceability criteria should always be independently checked. Thus, current AASHTO checks of member capacity (yielding or ultimate member capacity) and member behavior (cracking of concrete members and other member serviceability limits) are maintained in the present context of bridge redundancy.

The capacity of the structure to resist first member failure is expressed as the number of AASHTO HS-20 trucks that it can carry before this first member failure limit state is violated. This HS-20 load multiplier will be referred to as LF_1. LF_1 can be calculated by applying the dead loads and two AASHTO HS-20 vehicles on a linear elastic structural model of the bridge and then incrementing the loads until first member failure occurs. This calculation is performed by using a structural finite element analysis of the structure or the distribution factors of the LRFD specifications (*2*) but not the simplified AASHTO distribution factors given in the standard specifications (*1*).

2.3.2 Ultimate Capacity

The ultimate capacity limit is defined as the maximum possible truck load that can be applied on the structure before it collapses. Collapse is defined as the formation of a collapse mechanism or the point at which the structure is subjected to high levels of damage. A mechanism is the point at which the structure exhibits infinitely large levels of displacements, rendering it unusable. In this section, damage is defined as the crushing of the concrete of main members or, more generally, the loss in the load-carrying capacity of a main member. The load factor (HS-20 load multiplier) that corresponds to the ultimate limit state will be referred to as LF_u. LF_u can be calculated by analyzing the structure under the effect of the dead loads and two AASHTO HS-20 vehicles on a nonlinear structural model of the bridge and then incrementing the truck loads until the system collapses.

2.3.3 Functionality Conditions

Under the effect of high levels of load, the bridge structural system may be subjected to large levels of permanent deformations that may not necessarily lead to collapse but may render the bridge unsafe for regular traffic. This phenomenon will be referred to as the loss of functionality. Possible system functionality criteria include a maximum level of displacement or an allowable hinge rotation. The latter criterion is used in the 1996 AASHTO ASD (Allowable Stress Design) specifications (*1*). A displacement check is preferred here because hinge rotations are applicable only to girder bridges. For example, displacements are appropriate for all types of bridge structures, including trusses. The published literature does not provide any information on the level of displacement that can be tolerated before inadequate traffic conditions are encountered. Also, because most practice uses linear elastic analysis rather than nonlinear behavior, specific guidance is not available from current specifications either. The present 1996 AASHTO displacement limit (span length/1,000) is intended to provide member serviceability and is not related to the system's performance.

In a separate study on the inelastic rating of bridges, Galambos et al. (*4*) proposed using a maximum inelastic displacement equal to span length/300 as a limit state. This selection is based on "visible" permanent deformations. Following the same approach, this study proposes using the total live load displacement as a functionality criterion. A value equal to span length/100 as a displacement limit is proposed. This limit is believed to be the maximum visible displacement that a bridge user or an observer can tolerate. The displacement limit proposed is based on best engineering judgment and is consistent with displacement levels used by other researchers and engineers. For example, many of the full-scale bridge tests described in the literature were not continued until the actual collapse of the structure but were often stopped when the researchers perceived that the displacements reached "dangerously high levels." A review of such experiments (see Appendixes B and C) found that such perceived "dangerous levels" often corresponded to displacements on the order of span length/150 to span length/100. Also, the calculations performed in this study found that using a span length/100 as a functionality criterion produces results that are less sensitive to the modeling assumptions and the design details (see Appendixes B and C). Because the final recommendations are calibrated on the basis of the reliability and the performance of existing structures, these final recommendations will not be sensitive to the actual displacement criterion chosen.

The capacity of a structure to resist the violation of the maximum displacement (functionality) limit can be expressed as the number of AASHTO HS-20 trucks that can be placed on the structure before this functionality limit state is reached. LF_f is defined as the HS-20 load multiplier that will cause the violation of the functionality limit state accounting for the nonlinear behavior of the bridge members. LF_f can be calculated by analyzing the structure under the effect of the dead loads and two AASHTO HS-20 vehicles on a nonlinear structural model of the bridge and then incrementing the truck loads until a maximum displacement equal to the span length/100 is observed. Because redundancy is concerned with the performance of the structure, the displacements are checked in the main members only. The displacements of the slab or secondary members are not checked for this functionality limit state.

2.3.4 Damaged Conditions

The damaged bridge condition consists of the removal from the structural model of a main load-carrying component that might be subject to brittle fracture or to accidental loss of capacity due to collisions or to causes other than the application of the traffic load. The HS-20 load multiplier corresponding to the ultimate capacity of the damaged structure is defined as LF_d. LF_d can be calculated by analyzing the damaged structure under the effect of the dead loads and two AASHTO HS-20 vehicles on a nonlinear structural model of the bridge and then incrementing the truck loads until the structural system collapses.

2.3.5 Summary

In summary, the four limit states identified are defined as

- **Member failure,** which is a check of individual member safety using elastic analysis and member capacity as defined in current specifications;
- **Ultimate limit state,** which is defined as the ultimate capacity of the bridge system or the formation of a collapse mechanism;
- **Functionality limit state,** which is defined as the capacity of the structure to resist a live load displacement in a main longitudinal member equal to the span length/100; and
- **Damaged condition limit state,** which is defined as the ultimate capacity of the bridge system after the removal of one main load-carrying component from the structural model.

Values of LF_1, LF_u, LF_f, and LF_d, which give measures of the capacity of the structure for the four limit states listed above, are calculated in Appendixes B, C, and D for typical configurations of prestressed concrete, I-beam bridges; prestressed concrete, spread box-girder bridges; multibox beam bridges; and steel I-beam bridges.

Because redundancy is defined as the capability of the structure to continue to carry loads after the failure of one main member, a comparison of LF_u, LF_f, LF_d, and LF_1 would provide a measure of the level of bridge redundancy. The system reserve ratios for the ultimate limit state R_u, for the serviceability limit state R_f, and for the damaged condition R_d are defined as

$$R_u = \frac{LF_u}{LF_1}$$

$$R_f = \frac{LF_f}{LF_1}$$

$$R_d = \frac{LF_d}{LF_1} \qquad (1)$$

The system reserve ratios R_u, R_f, and R_d are nominal (deterministic) measures of bridge redundancy. For example, when the ratio R_u is equal to 1.0 ($LF_u = LF_1$), the ultimate capacity of the bridge system is equal to the capacity of the bridge to resist failure of its most critical member; such a bridge is nonredundant. As R_u increases, the level of bridge redundancy increases. Similar observations can be made about R_f and R_d. These two ratios may under certain circumstances, however, have values less than 1.0. A value of R_f less than 1.0 means that the bridge will exhibit a deformation equal to span length/100 at a load level smaller than the load that will cause the first member failure. This situation might occur in certain bridges because LF_1 is calculated with a linear elastic model, whereas LF_f accounts for the nonlinear behavior of the bridge. Similarly, R_d less than 1.0 means that a damaged bridge will be able to carry less live load than the load that will cause the first member failure in the intact linear elastic structure.

To check whether a bridge system has adequate levels of redundancy, it is sufficient to use a structural analysis program to calculate LF_u, LF_f, LF_d, and LF_1 and to verify that R_u, R_f, and R_d are adequate. Minimum acceptable values of R_u, R_f, and R_d should be established by examining the results of bridges that are clearly redundant according to current engineering practice. These minimum acceptable values should account for the uncertainties associated with determining the loads and the resistances of bridge superstructures.

Structural reliability methods have been developed to account for load and resistance uncertainties but may be more complicated than deterministic methods for practical implementation on a regular basis. To facilitate the implementation of reliability methods, code writing groups have bridged the gap between reliability theory and the deterministic approach by calibrating design and evaluation codes that provide uniform levels of reliability. This technique, known as the level I reliability method, was used in the development of the AASHTO LRFD specifications (*2,3*). In level I methods, the reliability model is transparent to the end user of the code. Whereas the load and resistance factors are calibrated based on reliability models, the end user of the code performs a deterministic check of the member safety by using these load and resistance factors without referring to reliability theory.

This study proposes a simple approach to develop reliability-based measures of bridge redundancy. These measures are then used to calibrate a deterministic (level I) format that implicitly accounts for the resistance and load uncertainties. To perform the reliability calculations, the maximum live loads expected to be applied on the bridge superstructure in its lifetime should be defined. Section 2.4 presents a discussion of the live load models used during the calibration process.

2.4 EXPOSURE PERIOD AND APPLIED LIVE LOADS

As mentioned previously, to be safe, a bridge: (a) should provide a reasonable safety level against first member failure; (b) should not reach its ultimate system capacity under extreme loading conditions; (c) should resist large deformations under expected traffic load conditions; and (d) should be able to carry some traffic loads after damage or the loss of a component. Two traffic conditions are recognized: extreme loading and regular loading.

2.4.1 Extreme Loading Conditions

In addition to carrying its dead load, a bridge should be able to carry the maximum truck load expected to be applied on it in its lifetime without reaching its ultimate capacity. This maximum expected lifetime load is a statistical variable that depends on the number of trucks that simultaneously cross the bridge, the positions of the trucks on the bridge deck, the weights of the trucks, the distribution of the weights to the individual axles, and the trucks' axle configurations. In addition, the load is a function of the dynamic impact. The design life of a bridge is normally equal to 75 years. For longer expected bridge lifespans, there is a higher probability of having heavier trucks simultaneously on the bridge. The 75-year lifespan, however, seems to provide an asymptotic limit beyond which the increase in the maximum expected load is practically negligible (*3*). The 75-year exposure period is used here for the ultimate limit state, the functionality limit state, and the first member failure limit state. In the latter case, the 75-year exposure period is consistent with the basis for the AASHTO LRFD specifications (*2*). The maximum expected lifetime load can be expressed in terms of equivalent AASHTO HS-20 truck loads. For example, LL_{75} is defined as the multiple of two HS-20 vehicles needed to produce the same load effect as the maximum expected lifetime load. This maximum lifetime load corresponds to the HL 93 loading used in the LRFD specifications. Table 1 gives the expected LL_{75} values for simple-span bridges of different span lengths for two-lane loadings. In this study, the two-lane loading is used as the reference load configuration. The results obtained from the two-lane loads can be extended to other numbers of lanes as will be discussed in Section 2.7.

The LL_{75} values in Table 1 are the same values used by Nowak (*3*) in the calibration of AASHTO's LRFD code. Table 1 also shows the coefficients of variation (COV) of the LL_{75} values. The COV is the ratio of the standard deviation of LL_{75} divided by the mean value. The COV values in Table 1 are also taken from Nowak (*3*). These COV values are used in this study to be consistent with the values used during the calibration of the AASHTO LRFD code. The final results obtained are robust, even if the actual COV depends on the span length (*5*).

TABLE 1 Mean and COV of applied loads as function of the effect of two side-by-side AASHTO HS-20 trucks

Span length (ft)	LL_{75} (*3*)	LL_2 (*3*)	V_{LL} (*3*)
45	1.67	1.53	19%
60	1.72	1.60	19%
80	1.81	1.67	19%
100	1.89	1.75	19%
120	1.98	1.84	19%
150	2.01	1.87	19%

These LL_{75} and LL_2 values given as a function of two HS-20 load effects.

2.4.2 Regular Traffic Conditions

Regular traffic loads are defined as recurrent loads expected to be regularly applied on the bridge. To determine the loads expected under regular traffic conditions, a 2-year exposure period is used. The 2-year exposure period is chosen because it corresponds to the biennial mandatory bridge inspection period. Thus, a 2-year exposure period is used for the damaged bridge condition because, even if damage (e.g., a fatigue fracture) goes unnoticed for a short period of time, it is bound to be discovered during inspection. LL_2 is defined as the number of two AASHTO HS-20 trucks needed to produce the same load effect as that expected under regular truck traffic conditions. Table 1 gives the expected 2-year loads expressed as multipliers of the effect of two AASHTO HS-20 trucks. The LL_2 values in Table 1 are adopted from the work of Nowak (*3*). Because no statistical data on the COV of LL_2 are available, the same COVs are assumed to be valid for both LL_{75} and LL_2, although it is generally known that the shorter period is normally associated with higher COVs. This assumption should not alter the final results because the reliability calibration process is known to be a robust process in the sense that errors in the database do not influence the final results as long as the new design or evaluation procedures are calibrated to match current acceptable practice (*5*).

2.5 DEFINITION OF SAFETY CRITERIA

The four limit states should be checked to ensure the satisfactory and safe performance of any bridge system under extreme or regular loading conditions. "Adequate" safety margins should also be provided to account for the uncertainties associated with determining bridge system capacity as well as the uncertainties associated with determining the live load levels that will be applied on the bridge. Adequate safety margins can be determined using reliability-based techniques similar to those used in the development of AASHTO's LRFD specifications (*2,3*).

The measure of safety used in the development of AASHTO's LRFD specifications is the reliability index β (*3*). The reliability index can be used as a measure of the reliability of structural members as well as structural systems. The reliability index accounts for both the margin of safety implied by the design procedure and the uncertainties in estimating member strengths and applied loads. The reliability index can be directly related to the probability of failure as explained in Throft-Christensen and Baker (*6*). In an LRFD code, the member design criteria are calibrated to produce a uniform value of β for the full range of application. For example, the AASHTO LRFD code was calibrated to provide a reliability index approximately equal to 3.5 for bridge members using the database assembled in Nowak (*3*). The *Ontario Highway Bridge Design Code* (OHBDC) was also calibrated to provide a uniform member reliability index of 3.5 using a different database assembled in 1979 (*7*). On the other hand, current 1996 AASHTO WSD and LFD criteria do not usually produce uniform levels of reliability indices (*5,8*).

Assuming that the member resistance represented by the load factor LF_1 and the applied maximum lifetime live load represented by the factor LL_{75} are random variables that follow lognormal distributions, then the reliability index β_{member} for the failure of the first member can be expressed using a lognormal format as

$$\beta_{member} = \frac{\ln \dfrac{\overline{LF}}{\overline{LL}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{2}$$

where $\overline{LF_1}$ is the mean value of the load factor that will cause the first member failure in the bridge, assuming elastic analysis (*6*). As explained in Section 2.3, LF_1 is related to the unfactored live load margin (R − D). Thus, $\overline{LF_1}$, which is the mean value of LF_1, relates to the strength capacity of the member represented by the resistance R and the dead load D. $\overline{LL_{75}}$ is the mean value of the maximum expected lifetime live load, including dynamic load allowance effect. V_{LF} is the coefficient of variation of LF_1, while V_{LL} is the coefficient of variation of the maximum expected live load LL_{75}. The denominator in Equation 2 gives an overall measure of the uncertainty in estimating the resistance, the dead load, and the live load including dynamic impact.

Nowak (*3*) suggests that the maximum live load follows a Gumbel distribution. If the live load follows a Gumbel distribution rather than a lognormal distribution, then Equation 2 is no longer valid and the calculation of the reliability index requires a "level II reliability" program (*6*). The level II program maps the distribution of each random variable into an equivalent normal distribution at the point where the failure is most likely to occur and can be used to model any type of probability distribution. An example in Appendix C demonstrates that the results of level II calculations with a Gumbel distribution for the live load are similar to those of Equation 2, which assumes a lognormal distribution. The rest of this chapter demonstrates the proposed calibration procedure using a lognormal format for illustrative purposes only. The actual reliability calculations performed in the appendixes use a Gumbel distribution for the live load, and the reliability indices are obtained using a level II program.

Assuming that the load factor LF_u and the live load factor LL_{75} follow lognormal distributions, the reliability index of the system for the ultimate limit state can be defined as

$$\beta_u = \frac{\ln \dfrac{\overline{LF_u}}{\overline{LL_{75}}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{3}$$

where $\overline{LF_u}$ is the mean value of the load factor corresponding to the ultimate limit state. $\overline{LF_u}$ relates to the strength capacity of the system and the dead load. $\overline{LL_{75}}$ and V_{LL} are the same values used to calculate β_{member}. Because of lack of data on the coefficients of variation of the bridge systems, it is assumed that LF_u, LF_f, and LF_d have the same coefficients of variation V_{LF} used for LF_1. The theory of reliability of structural systems demonstrates that, in general, the COV of a system is smaller than the COV of the individual members. However, this observation is based on the assumption that the structural model used during the nonlinear analysis is exact. In this study, the COVs of the system capacity (LF_u, LF_f, and LF_d) are assumed to be equal to the COV of the member capacity (LF_1) to account for the modeling uncertainties. Equation 3 assumes that the live load (LL_{75}) follows a lognormal distribution. If the load follows a Gumbel distribution as suggested by Nowak (*3*), then a level II program must be used.

For the functionality limit state, the performance of the complete system can also be measured as a system serviceability reliability index β_{funct} defined as

$$\beta_{funct} = \frac{\ln \dfrac{\overline{LF_u}}{\overline{LL_{75}}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{4}$$

where $\overline{LF_f}$ is the mean load factor to reach the serviceability limit state. $\overline{LF_f}$ is the capacity of the bridge system to resist large deformations and the applied dead load, and $\overline{LL_{75}}$ is the mean maximum applied live load for the life of the structure. Equation 4 assumes that the load follows a lognormal distribution; if the load follows a Gumbel distribution, then a level II program must be used.

Finally, assuming a lognormal model, the system's ability to sustain loads after damage can be expressed as a system reliability index for damaged conditions, $\beta_{damaged}$, defined as

$$\beta_{damaged} = \frac{\ln \dfrac{\overline{LF_d}}{\overline{LL_2}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{5}$$

where $\overline{LF_d}$ is the mean load factor to reach the ultimate capacity of the damaged system. $\overline{LF_d}$ is the capacity of the system after one member is damaged and the dead load. A 2-year exposure period is used for the damaged conditions. The mean live load for the 2-year period is expressed as $\overline{LL_2}$, which is a multiplier of the effects of two HS-20 vehicles. Equation 5 assumes that the load follows a lognormal distribution. If the load follows a Gumbel distribution, then a level II program must be used.

To study the redundancy of a system, it is useful to examine the difference between the reliability indices of the system expressed as β_{ult}, β_{funct}, and $\beta_{damaged}$ and the reliability index of the most critical member expressed as β_{member}. The relative reliability indices are defined as

$$\Delta\beta_u = \beta_{ult} - \beta_{member}$$
$$\Delta\beta_f = \beta_{funct} - \beta_{member}$$
$$\Delta\beta_d = \beta_{damaged} - \beta_{member} \tag{6}$$

These relative reliability indices give measures of the relative safety provided by the bridge system compared with the nominal safety of first member failure. It is proposed to use these relative reliability indices to provide reliability-based measures of redundancy. Thus, a bridge system will provide adequate levels of system redundancy if the relative reliability indices are adequate.

Some codes, such as the LRFD code, are calibrated to provide uniform levels of member reliability indices. In this case, providing adequate system reliability indices is identical to providing adequate values of relative reliability indices. Therefore, checking the values of β_{ult}, β_{serv}, and $\beta_{damaged}$ is sufficient to verify that adequate levels of system redundancy are present. On the other hand, when the member reliability indices are not uniform (e.g., when using WSD or LFD criteria), checking the values of β_{ult}, β_{funct}, and $\beta_{damaged}$ is not sufficient to verify that adequate levels of system redundancy are present. In this case, $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ should be checked. A review of existing designs should be undertaken to determine the values of $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ that are required to provide adequate levels of system redundancy.

Substituting Equations 3 through 5 into Equation 6, the relative reliability indices for a lognormal model become

$$\Delta\beta_u = \frac{\ln\frac{\overline{LF_u}}{\overline{LL_{75}}} - \ln\frac{\overline{LF_1}}{\overline{LL_{75}}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} = \frac{\ln\frac{\overline{LF_u}}{\overline{LF_1}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{7}$$

$$\Delta\beta_f = \frac{\ln\frac{\overline{LF_f}}{\overline{LL_{75}}} - \ln\frac{\overline{LF_1}}{\overline{LL_{75}}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} = \frac{\ln\frac{\overline{LF_f}}{\overline{LF_1}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{8}$$

and

$$\Delta\beta_d = \frac{\ln\frac{\overline{LF_d}}{\overline{LL_{75}}} - \ln\frac{\overline{LF_1}}{\overline{LL_2}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} = \frac{\ln\frac{\overline{LF_d}\,\overline{LL_{75}}}{\overline{LF_1}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{9}$$

Equations 7 through 9 show that, for a lognormal model, the relative reliability indices are related to the system reserve ratios LF_u/LF_1, LF_s/LF_1, and LF_d/LF_1 defined in Equation 1 as R_u, R_f, and R_d. Appendixes B and C show that if LF_1 is changed by a certain factor, then LF_u, LF_f, and LF_d would change by approximately the same percentage. Hence, assuming that $\overline{LF_u}/\overline{LF_1} = LF_u/LF_1$ and assuming that $\overline{LF_f}/\overline{LF_1} = LF_f/LF_1$ and $\overline{LF_d}/\overline{LF_1} = LF_d/LF_1$, Equations 7 through 9 become

$$\Delta\beta_u = \frac{\ln R_u}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{7a}$$

$$\Delta\beta_f = \frac{\ln R_f}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{8a}$$

$$\Delta\beta_d = \frac{\ln R_d \frac{\overline{LL_{75}}}{\overline{LL_2}}}{\sqrt{V_{LF}^2 + V_{LL}^2}} \tag{9a}$$

Thus, assuming a lognormal model, the reliability-based measures of redundancy as defined by Equation 6 are related to the nominal (deterministic) measures of redundancy defined in Equation 1. If a Gumbel distribution is assumed for the live load model, then Equations 7 through 9 are no longer valid, and the $\Delta\beta$ values must be calculated from the level II program. Nevertheless, the correlation between the $\Delta\beta$ and the LF/LF_1 ratios will always be maintained. Calculations of $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ for typical bridge configurations have been performed using a Gumbel distribution for the live load as described in Appendixes B, C, and D for prestressed concrete, I-beam bridges; for prestressed concrete, spread box-girder and multibox beam bridges; and for steel I-beam bridges.

In the next section, a reliability-based calibration is performed to determine the minimum values of relative bridge capacities (i.e., the ratio of system capacity with respect to member capacity) represented by R_u, R_f, and R_d that are required to ensure an adequate level of bridge redundancy. Target $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ values are obtained from reviewing the performance of typical bridge design as will be discussed in Section 2.6.

2.6 DETERMINATION OF TARGET RELIABILITY INDICES

The object of this study is to calibrate a set of system factors that can be used with the normal design equations to account for the redundancy of typical bridge superstructures. In addition, this study proposes to calibrate a step-by-step procedure to check the redundancy of nontypical structures using a nonlinear finite element analysis. To perform the calibration of the system factors and the step-by-step procedure, minimum target $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ values that a bridge should satisfy must be obtained. In this study, these target values are extracted on the basis of a review of the performance of existing redundant designs. This section describes how this selection process was performed. Appendixes B, C, and D give more detailed information on the calibration process.

To perform the reliability calibration, a large number of common-type multigirder steel bridges; prestressed concrete, I-beam bridges; prestressed concrete, spread box-girder bridges; and prestressed concrete, multibox beam bridges were analyzed. The results of these analyses are given in Appendixes B, C, and D. Values of LF_1, LF_u, LF_f, and LF_d were calculated for each bridge using the *non*linear *b*ridge *an*alysis program, NONBAN, developed in this study. Appendix F describes the program NONBAN, which accounts for the nonlinear behavior of steel and concrete bridges. Methods to obtain the member nonlinear material properties that are used with NONBAN are described in Appendixes B, C, and D for each bridge type. NONBAN was used in this study for convenience; however, other commercially available nonlinear finite element packages could have been used instead. Appendix F lists a few other packages that can be used. NONBAN was developed to simplify the input data requirement normally associated with the performance of nonlinear analyses. A full description of the program and its capability is provided in Appendix F. The validity of the program was extensively tested as shown in Appendixes B, C, D, and F.

Given the load factors LF_1, LF_u, LF_f, and LF_d, the reliability indices β_{member}, β_{ult}, β_{funct}, and $\beta_{damaged}$ were calculated for each bridge configuration using a level II reliability program. In addition, $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$ were calculated for each bridge using Equation 6. Appendixes B, C, and D give detailed descriptions of the bridges analyzed and the results obtained. The extraction of the target relative reliability indices is performed for the ultimate limit state, functionality limit state, and the damaged condition on the basis of bridge designs that are known to be redundant as described below.

2.6.1 Ultimate Limit State

In current practice, all two-girder bridges, and according to several opinions even three-girder bridges, are defined as nonredundant. On the other hand, all bridges with four or more beams are always classified as redundant. Therefore, it is recommended to use the average $\Delta\beta_u$ value obtained from four-beam bridges as the target relative reliability index that all bridges should satisfy to provide adequate levels of redundancy. The calculations given in Appendixes B and C show that typical two-lane, simple-span, steel I-beam bridges with four beams at spacings greater than 4 ft have $\Delta\beta_u$ values that range from 0.46 to 0.94 with an average of 0.72. The simple-span, two-lane prestressed concrete I-beam bridges with four beams at spacings greater than 4 ft produce $\Delta\beta_u$ values that range from 0.70 to 1.28 with an average of 0.97. Therefore, it is proposed to use a $\Delta\beta_u$ value equal to 0.85 as target for the ultimate limit state of redundant bridges. The 0.85 value is the average of the steel bridges and prestressed concrete, I-beam bridges.

2.6.2 Functionality

Bridges that may have very high ultimate capacities may reach these ultimate capacities only after extensive deformations rendering these bridges unfit for use at loads lower than ultimate capacity. For this reason, a functionality check is required as part of this redundancy study. The functionality check is performed by extracting a target $\Delta\beta_f$ based on typical bridge configurations.

Appendixes B and C have shown that steel bridges with four beams at spacings greater than 4 ft produce an average $\Delta\beta_f$ value equal to 0.53 with a range from 0.41 to 0.62. The prestressed concrete bridges produce softer responses than steel bridges. The average $\Delta\beta_f$ for the prestressed concrete bridges with four beams at spacings greater than 4 ft is about 0.0 with a range from -0.17 to 0.41. Negative values of $\Delta\beta_f$ indicate that, in many instances, the nonlinear behavior under heavy loads may cause many bridge systems to have a higher risk of exceeding the functionality limit than the risk of having a failure in one member. Therefore, it is important to control the deformations of such systems by imposing a penalty on bridges that may have high levels of redundancy but produce low levels of functionality. For this reason, it is recommended to use a $\Delta\beta_f$ value of 0.25 as the target relative reliability index value for the functionality limit state. This value is an average value from the two-lane, four-beam bridges for both prestressed concrete and steel bridges.

2.6.3 Damaged Condition

Damaged two-lane bridges gave values of $\Delta\beta_d$ that range between -5.00 and -1.15 with an average of -2.96 for two-lane, simple-span steel bridges with four beams at spacings greater than 4 ft or higher. The prestressed concrete, simple-span, two-lane I-beam bridges with four beams at spacings of 4 ft or higher produced $\Delta\beta_d$ that range from -4.79 to -0.9 with an average of -2.40. Based on theses results, it is recommended to use a $\Delta\beta_d$ value of -2.70 as the target relative reliability index for the damaged condition. This -2.70 value is the average value obtained for both steel and prestressed concrete I-beam bridges with four beams.

2.6.4 Summary

In summary, the conclusions obtained for the typical bridge configurations studied in Appendixes B, C, and D, for simple-span, I-beam prestressed concrete and steel bridges revealed that a bridge will provide adequate levels of redundancy if all the following conditions are satisfied:

- It gives a value of $\Delta\beta_u$ greater than or equal to 0.85.
- It gives a value of $\Delta\beta_f$ greater than or equal to 0.25.
- It gives a value of $\Delta\beta_d$ greater than or equal to -2.70.

2.6.5 Target System Reliability Indices

The database used in this study showed that two-lane bridges designed to satisfy the 1996 AASHTO LFD criteria with HS-20 loading (*1*) are adequately redundant if $\Delta\beta_u$ is greater than 0.85. If the AASHTO LRFD code is calibrated to produce a member reliability index of 3.5, then an adequate redundant bridge built to satisfy the LRFD criteria should also produce a β_{ult} value greater than or equal to 4.35. Similarly, a β_{funct} greater than or equal to 3.75 is required to satisfy the functionality criterion. Finally, $\beta_{damaged}$ greater than or equal to 0.8 is required to satisfy the damaged condition criterion. A reliability index of 0.8 indicates that the probability of failure of a bridge that has completely lost the load-carrying capacity of one main member is only 21.2 percent.

The reliability indices are calculated using the database assembled in this study as explained in Appendixes B, C, and D. Moses and Ghosn (*5*), however, have shown that reliability-based code calibrations are insensitive to the database used as long as the target reliability indices are extracted based on the performance of adequate existing designs. Because the procedures and the tables developed in the subsequent sections are calibrated to achieve the target reliability indices with the same database used to extract the target values, it is concluded that the proposed tables will be insensitive to variations in the database.

2.7 IMPLEMENTATION: DIRECT REDUNDANCY CHECK

The direct redundancy check procedure proposed in this section is based on satisfying minimum values of the relative reliability indices $\Delta\beta_u$, $\Delta\beta_f$, and $\Delta\beta_d$.

The conclusions in Section 2.6 revealed that bridge systems that have adequate levels of redundancy produced relative system reliability indices $\Delta\beta_u$ equal to 0.85 or higher; $\Delta\beta_f$ equal to 0.25 or higher; and $\Delta\beta_d$ equal to -2.7 or higher. Thus, a bridge will provide adequate levels of redundancy if all the following conditions are satisfied:

- It gives a value of $\Delta\beta_u$ greater than or equal to 0.85.
- It gives a value of $\Delta\beta_f$ greater than or equal to 0.25.
- It gives a value of $\Delta\beta_d$ greater than or equal to -2.70.

The values of the system reserve ratios $R_u = LF_u/LF_1$, $R_f = LF_f/LF_1$, and $R_d = LF_d/LF_1$ that are required to satisfy a minimum level of bridge redundancy can be calculated using Equations 7 through 9 if the live load follows a lognormal distribution. When the load follows a Gumbel distribution, the required values for R_u, R_f, and R_d are obtained from the level II reliability program. The reliability program applied on all the simple-span I-beam bridges (prestressed concrete and steel) produced an average required R_u value of 1.28. The required R_f value is, on the average, 1.08. The required R_d value is 0.47. If Equations 7 through 9 are used instead of the level II reliability program, the required values for R_u, R_f, and R_d are 1.22, 1.06, and 0.48, respectively, which are reasonably close to those found from the level II reliability program. The value of V_{LF} used with Equations 7 through 9 is 14.2 percent, which is the average value of V_{LF} found for all of the simple-span bridges analyzed, while the value of V_{LL} used is 19 percent, as suggested by Nowak (*3*).

On the basis of the results of the reliability analysis of the typical simple-span configurations studied in Appendixes B and C, a bridge system is considered adequately redundant if the analysis of the bridge produces a system reserve ratio:

- For the ultimate capacity R_u greater than or equal to 1.30;
- For the functionality limit state R_f greater than or equal to 1.10; and
- For the damaged conditions R_d greater than or equal to 0.50.

These required values of system reserve ratios are summarized in Table 2. These values are obtained by rounding up the values calculated from the level II reliability program. The required $R_{u\,req}$ value is 1.30, which means that R − D at collapse should be 30 percent higher than R − D when first member failure occurs. The required $R_{f\,req}$ value is 1.10, which means that the live load margin (R − D) of the system when the maximum displacement of span length/100 is reached should be 10 percent higher than the live load margin when first member failure occurs. Finally, the required $R_{d\,req}$ value of 0.50 indicates that a damaged bridge must be able to carry approximately 50 percent of the live load that an intact bridge should carry before first member failure occurs. A value of R_d less than 1.0 means that a damaged bridge will be able to carry less live load than the load that will cause the first member failure in the intact structure. A particular bridge system will provide adequate levels of redundancy if the values of R_u, R_f, and R_d calculated for that bridge are higher than the required values given here. First member failure is always defined using linear elastic analysis procedures that are consistent with current load evaluation methods while system capacity for both intact and damaged conditions is defined using nonlinear system analysis methods.

The check of R_u, R_f, and R_d is a check on the redundancy of the system. Bridges that are not redundant may still pro-

TABLE 2 Required load factor ratios for direct system redundancy approach

Ultimate Limit State:	$R_{u\,req} = (LF_u/LF_1)_{req}$	1.30
Functionality Limit State:	$R_{f\,req} = (LF_f/LF_1)_{req}$	1.10
Damaged Condition:	$R_{d\,req} = (LF_d/LF_1)_{req}$	0.50

These values are valid for any number of lanes and for any load model including HS-20 or HS-25 trucks.

vide high levels of system safety if their members are overdesigned. Therefore, the redundancy check should always be performed in conjunction with a member safety check. This is achieved by comparing the actual capacity of the bridge members to the capacity required by the specifications. In this case, R_{req} is defined as the member capacity required to satisfy AASHTO's specifications (*1,2*). Any acceptable member design criteria can be used. For example, the required member capacity R_{req} is calculated for the most critical member using AASHTO's design and evaluation equation:

$$\phi R_{req} = \gamma_d D_n + \gamma_l L_n (1 + I) \tag{10}$$

where ϕ is the resistance factor, γ_d is the dead load factor, γ_l is the live load factor, D_n is the nominal or design dead load, and L_n (1 + I) is the nominal or design live load including impact. Equation 10 has a general format that can be used for any AASHTO criteria. For example, for WSD criteria, γ_d and γ_l are given as 1.0 and ϕ is equal to 1/0.55. For the LFD criteria, ϕ depends on the type of member being analyzed, γ_d is equal to 1.3, and γ_l is 2.17. For the LRFD criteria, Equation 10 assumes that the load modification factor is equal to 1.0. In LRFD, the ϕ factor depends on the type of material, and γ_d depends on the type of dead load. For example, $\gamma_d = 1.25$ is used for component dead load, and γ_l depends on the load combination used. For example, $\gamma_l = 1.75$ is used for the base load combination case.

The required member load factor $LF_{1\,req}$ is defined as

$$LF_{1\,req} = \frac{R_{req} - D}{L_{HS\text{-}20}} \tag{11}$$

where R_{req} is the required member capacity obtained from Equation 10, D is the dead load effect on the most critically loaded member, and $L_{HS\text{-}20}$ is the effect of the AASHTO HS-20 vehicle load on the most critical member. $L_{HS\text{-}20}$ is calculated from a linear elastic analysis of the bridge rather than the simplified formula for distribution factor provided by the 1996 AASHTO specifications (*1*). As in the case of LF_1, the calculation of $LF_{1\,req}$ is performed with the HS-20 trucks without including the impact factor. To provide a measure of the adequacy of the actual member capacity represented by LF_1 to that required by the AASHTO specifications, the member reserve ratio r_1 is defined as

$$r_1 = \frac{LF_1}{LF_{1\,req}} = \frac{R_{provided} - D}{R_{req} - D} \tag{12}$$

where $R_{provided}$ is the provided member capacity, R_{req} is the required member capacity, and D is the dead load.

Bridge members that are designed to exactly match the AASHTO specifications will produce a member reserve ratio of 1.0. Members that are overdesigned will produce r_1 values greater than 1.0. This member evaluation procedure can be used with any bridge design criteria, including WSD, LFD, and LRFD or bridge evaluation procedures, including operating and inventory rating levels using HS-20 or any other appropriate loading.

The member reserve ratio is used in conjunction with a check of R_u, R_f, and R_d to recommend system factors using the redundancy check concept as outlined in the next section.

2.7.1 Step-by-Step Procedure

This section presents a direct method to determine the redundancy level of a bridge system using a detailed nonlinear finite element analysis. The procedure can be used in conjunction with any member checking criteria including AASHTO's WSD, LFD, or LRFD including either inventory or operating ratings. The basic procedure has been calibrated on the basis of providing adequate redundancy levels for two-lane bridges, although it is applicable to any number of lanes as will be discussed below. The following 12 steps are involved in the analysis of the redundancy of a given bridge superstructure.

Step 1. Use the AASHTO specifications to find the required member capacity R_{req} for the bridge members using Equation 10.

Step 2. Develop a structural model of the bridge to be used with a finite element package that allows static nonlinear analysis of structure. In the model, use the best estimate of the nonlinear material properties of the structural members without applying any safety factors or strength reduction factors. Use a best estimate for the material properties. Apply a best estimate of the unfactored dead load.

Step 3. Identify lateral and longitudinal loading positions on the structure for the HS-20 AASHTO trucks to produce the most critical loading effect. Do not include impact factors.

Step 4. Apply the loads to the structure and perform a linear elastic analysis to calculate $L_{HS\text{-}20}$, which gives the effect of the AASHTO trucks on the most critical member. Then, using Equation 11, calculate $LF_{1\,req}$.

Step 5. Increment the loads of the HS vehicles until the first member reaches its limiting capacity. Note the load factor LF_1 by which the original HS-20 trucks are scaled for the first member failure to occur. Then, using Equation 12, calculate the member reserve ratio r_1.

Step 6. Using a nonlinear structural model and nonlinear material properties, increment the applied HS-20 truck loads until the maximum vertical deflection of a primary member reaches a deflection equal to span length/100. Note the load factor LF_f by which the original HS-20 trucks are scaled to achieve the span length/100 displacement level. In the ratio $R_f = LF_f/LF_1$, if the ratio is greater than 1.1, then the bridge has a sufficient level of redundancy to satisfy the functionality limit state. Calculate the redundancy ratio for functionality r_f:

$$r_f = \frac{R_f}{1.10} \tag{13}$$

Step 7. Increment the load further until the ultimate limit state is reached. The ultimate limit state is defined as the maximum possible truck load that can be applied on the structure before it collapses. Collapse is defined as the formation of a collapse mechanism or the point at which the structure is subjected to high levels of damage. A mechanism is the point at which the structure exhibits infinitely large levels of displacements rendering it unusable. In this section, damage is defined as the crushing of the concrete of main members or, more generally, the loss in the load-carrying capacity of a main member. The load factor calculated in this step is LF_u. If $R_u = LF_u/LF_1$ is larger than 1.30, then the bridge has a sufficient level of redundancy to satisfy the ultimate limit state. Calculate the redundancy ratio r_u:

$$r_u = \frac{R_u}{1.30} \tag{14}$$

Step 8. Identify members whose failure might be critical to the structural integrity of the bridge. These damage scenarios should be specified in consultation with the bridge owner. These members could be: (a) members that can be damaged by an accidental collision by a vehicle, ship, or debris; (b) prestressed concrete members that might loose prestressing due to fatigue of cables; or (c) steel members that are prone to fatigue or are fracture critical.

Step 9. Remove one of the members identified in Step 8 from the structural model and repeat the nonlinear analysis. Next, determine the load factor of the damaged bridge LF_d at ultimate. If the ratio $R_d = LF_d/LF_1$ exceeds 0.50, the bridge provides a sufficient level of redundancy. Finally, calculate the redundancy ratio for damaged conditions:

$$r_d = \frac{R_d}{0.50} \tag{15}$$

Step 10. Place the member removed in Step 9 back into the model and remove another critical member. Repeat Step 9 until all the critical members identified in Step 8 are checked.

Step 11. Repeat Steps 3 through 10 to cover all critical load patterns. The final values used for LF_u, LF_f, LF_d, and LF_1 are the minimum values obtained from checking different loading patterns.

Step 12. If all the redundancy ratios r_u, r_f, and r_d obtained from the analysis are larger than 1.0, then the bridge has a sufficient level of redundancy. If any redundancy ratio is less than 1.0, then the bridge does not have a sufficient level of redundancy and corrective measures should be undertaken to improve bridge safety. Corrective measures may include changing the bridge topology, strengthening the bridge members, or decreasing the rating of the bridge.

2.7.2 Generalizations

One-lane and multilane bridges are analyzed using the same step-by-step procedure outlined above with the appropriate number of AASHTO HS-20 trucks. Because the redundancy approach uses the normalized ratios of LF_u/LF_1, LF_f/LF_1, and LF_d/LF_1, the same procedure can be used for one-lane and multilane bridges without requiring any corrections to the $R_{u\,req}$, $R_{f\,req}$, and $R_{d\,req}$ values given in Table 2.

The analysis can also be performed for HS-25 trucks, or any other truck configuration using the same procedure. The required $R_{u\,req}$, $R_{f\,req}$, and $R_{d\,req}$ values given in Table 2 should not be modified.

The procedure is applicable for all bridge configurations including simple-span and continuous bridges and is also valid for all material types.

2.7.3 Redundancy Factors

The redundancy factor ϕ_{red} is defined as

$$\phi_{red} = \min\,(r_1 r_u,\ r_1 r_f,\ r_1 r_d) \tag{16}$$

where r_1 is the member reserve ratio defined in Equation 12, r_u is the redundancy ratio for the ultimate limit state defined in Equation 14, r_f is the redundancy ratio for the functionality limit state defined in Equation 13, and r_d is the redundancy ratio for the damaged condition defined in Equation 15. If ϕ_{red} is less than 1.0, it indicates that the bridge under consideration has an inadequate level of system redundancy. A redundancy factor greater than one indicates that the level of system safety is adequate.

To improve the redundancy of a bridge, the geometric configuration should be changed by adding members. If this cannot be achieved, nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations. By strengthening the members, an overall satisfaction of the system reliability targets is achieved. The member strengths of nonredundant bridges should be improved by increasing their member reserve capacities ($R - D$) by a factor equal to $1/\phi_{red}$ as shown:

$$R' - D' = \frac{R - D}{\phi_{red}} \tag{17}$$

where R' is the resistance required to satisfy the redundancy criteria proposed in this study. D' is the updated dead load corresponding to the member with resistance R'. R is the original resistance in the member, D is the original dead load in the member, and ϕ_{red} is the redundancy factor. In Equation 17, the ϕ_{red} factor is a penalty-reward factor whereby bridges with nonredundant configurations would be required to have higher member capacities than similar bridges with redundant configurations. On the other hand, redundant configurations will be rewarded by allowing their members to have lower capacities.

If the members of an existing bridge cannot be strengthened, then the bridge rating should be lowered by applying a rating factor as shown in Section 2.9.

To ensure that a minimum level of member safety is maintained, it is recommended that maximum r_u, r_f, and r_d values of 1.20 be used as upper limits (i.e., use 1.20 as a maximum limit on ϕ_{red} before applying r_1). The 1.20 limit is based on the maximum load modifier factor proposed in the AASHTO LRFD specifications (*2*). In fact, the LRFD specifications propose a minimum load modifier of $0.95 \times 0.95 \times 0.95$. This product produces a minimum value of 0.86, which is equivalent to a maximum redundancy factor of 1.17. On the other hand, the minimum value of ϕ_{red} proposed is 0.80; that is, the maximum penalty that a nonredundant bridge is assigned is 20 percent, while the maximum reward is also 20 percent. The 40 percent range is also on the same order of magnitude as the ratio of the WSD safety factor for operating rating to that of inventory rating (0.75/0.55). In current practice, operating ratings are often used during the strength evaluation of bridges that are known to be redundant. On the other hand, if the bridge is known to be nonredundant, it is common to use the inventory stress ratings. Therefore, using a maximum range of 0.4 on ϕ_{red} is consistent with current practice.

In principle, the same redundancy factor ϕ_{red} should be applied to all the members of the bridge system. In reality, some members may contribute less than other members toward the overall system capacity; therefore, using the same ϕ_{red} factor for all the members may be inefficient. To be more efficient, the redundancy factor ϕ_{red} may be applied to the most critical member(s) only and the full analysis described above is repeated until the system redundancy requirements are satisfied.

Applying a redundancy factor ϕ_{red} less than 1.0 will improve the bridge members' strengths represented by LF_1 and will also improve system strength expressed as LF_u, LF_f, and LF_d. Thus, the system ratios R_u, R_f, and R_d will practically remain unchanged, and a nonredundant bridge will remain nonredundant. However, by applying a redundancy factor ϕ_{red} less than 1.0, the reliability index for one member, as well as the system reliability indices, will be increased. Thus, nonredundant designs are penalized by requiring higher member safety levels than similar bridges with redundant configurations.

The procedure proposed in this section to perform the redundancy evaluation of bridge systems is illustrated in Chapter 3.

2.8 SYSTEM FACTORS FOR COMMON-TYPE BRIDGES

Section 2.7 provided a step-by-step procedure to evaluate the redundancy of bridge superstructures assuming that a program capable of performing a nonlinear incremental analysis is available. The step-by-step procedure is used to calculate the redundancy factor ϕ_{red}, which provides a measure of the redundancy of a bridge superstructure. Appendixes B, C, and D developed tables of system factors ϕ_s applicable to common-type multigirder steel and prestressed concrete I-beam, spread box-beam, and multibox beam bridges. The system factors are calibrated to be used in the design check equation as follows:

$$\phi_s \, \phi R' = \gamma_d D_n + \gamma_l L_n(1 + I) \tag{18}$$

where ϕ_s is the system factor. This is defined as a statistically based multiplier relating to the safety and redundancy of the complete system. The system factor is applied to the factored nominal member resistance. The system factor proposed replaces the load modifier η used in Section 1.3.2 of the LRFD Specifications. The factor is placed on the left side of the equation because the system factor is related to the capacity of the system and should be placed on the resistance side of the equation as is the norm in reliability-based calibration. ϕ is the member resistance factor, R' is the required resistance capacity of the member accounting for the redundancy of the system, γ_d is the dead load factor, D_n is the dead load effect, γ_l is the live load factor, L_n is the live load effect on an individual member, and I is the dynamic impact factor. When ϕ_s is equal to 1.0, Equation 18 becomes the same as the current design equation. If ϕ_s is greater than 1.0, this indicates that the system's configuration provides a sufficient level of redundancy. When it is less than 1.0, then the level of redundancy is not sufficient.

The approach used to develop the system factor tables is similar to the approach used in Section 2.7 to provide consistent levels of redundancy. The system factor tables developed in the appendixes are calibrated using a reliability model such that a system factor equal to 1.0 indicates that

- For the ultimate limit state, the bridge has an average relative reliability index $\Delta\beta_u$ equal to 0.85.
- For the functionality limit state, the relative reliability index $\Delta\beta_f$ is on the average equal to 0.25.
- For a damaged bridge, the relative reliability index $\Delta\beta_d$ is on the average equal to -2.70.

During the calibration process, the assumed damage scenarios consisted of the complete removal of a main girder.

The system factors for each bridge configuration considered in this study are given in the appendixes. For each configuration, three system factors are obtained corresponding to the three system limit states (ultimate, functionality, and damaged). The system factor that should be used is the minimum value. In addition, as recommended for the direct analysis system redundancy check, it is recommended to use a maximum system factor value of 1.20 and a minimum value of 0.80.

The system factors given in the appendixes are used for simple-span and continuous bridges with parallel members of the same capacity. Separate tables are provided for simple-span, prestressed concrete and steel I-beam bridges with 4, 6, 8, 10, and 12 beams (Appendixes B and C). Simple-span, prestressed concrete bridges with 2, 3, and 5 spread boxes as well as multibox beam bridges with up to 11 adjacent boxes are considered (Appendix D). Two-span, continuous I-beam bridges that are designed according to common U.S. practice were also analyzed (Appendixes B and C). For configurations that might be slightly different from those considered, it is possible to extract the system factors from the tables by interpolation.

Appendixes B and C performed sensitivity analyses using bridges with up to +/−50 percent of the required design strength. The results were found not to be very sensitive to such variations in member capacities; the corresponding change in the system factor was within +/−5 percent. Therefore, the system factor tables are applicable for bridges that satisfy any acceptable AASHTO design criteria including the LFD, WSD, and LRFD criteria. The tables are applicable for bridge designs that have R/D (resistance to dead load) ratios that fall within the range observed with current design standards. For bridges with R/D ratios that are much different from current designs, the step-by-step analysis procedure described in Section 2.7 should be used.

2.8.1 System Factor Tables

The results of the system factors are summarized in Tables 3 and 4 for simple-span, steel I-beam bridges and simple-span, prestressed concrete I-beam bridges. Tables 5 and 6 summarize the results for continuous span steel and continuous span prestressed concrete bridges. Table 5 is applicable to bridges with compact sections in the negative bending region.

The calculations performed in Appendix C have shown that continuous steel bridges with noncompact sections in negative bending are nonredundant. The analysis of noncompact sections performed in Appendix C assumes that a noncompact member will immediately start shedding its load as soon as its maximum bending capacity is reached (i.e., no member ductility is provided). Depending on the design

TABLE 3 System factors for redundancy of simple-span, steel I-beam bridges

Spacing		4 Beams	6 Beams	8 Beams	10 Beams
4ft	ultimate	0.86	1.03	1.05	1.05
	functi.	0.96	1.09	1.11	1.11
	damage	1.22	1.44	1.46	1.46
6 ft	ultimate	0.97	1.01	1.01	1.01
	functi.	1.05	1.08	1.08	1.08
	damage	1.20	1.27	1.27	1.27
8 ft	ultimate	0.99	1.00	1.00	1.00
	functi.	1.06	1.08	1.08	1.08
	damage	1.05	1.09	1.09	1.09
10 ft	ultimate	0.98	0.99	0.99	
	functi.	1.06	1.08	1.08	
	damage	0.88	0.90	0.90	
12 ft	ultimate	0.96	0.97		
	functi.	1.06	1.06		
	damage	0.70	0.76		

For each configuration, use the lowest value from the ultimate limit state, functionality limit state and damaged condition.

The values shown in the tables for the damaged limit states shall be increased by 0.10 for bridges provided with a distributed set of diaphragms.

A minimum value of 0.80 shall be used.

A maximum value of 1.20 shall be used.

TABLE 4 System factors for redundancy of simple-span, prestressed concrete I-beam bridges

Spacing		4 Beams	6 Beams	8 Beams	10 Beams
4ft	ultimate	0.87	1.04	1.08	1.08
	functi.	0.89	0.99	1.00	1.01
	damage	1.11	1.36	1.36	1.33
6 ft	ultimate	0.98	1.06	1.06	1.06
	functi.	0.96	0.98	0.99	1.00
	damage	1.21	1.25	1.26	1.26
8 ft	ultimate	1.04	1.07	1.07	1.07
	functi.	0.95	0.98	0.99	1.00
	damage	1.13	1.18	1.18	1.18
10 ft	ultimate	1.06	1.06	1.06	
	functi.	0.95	0.98	0.98	
	damage	1.05	1.07	1.07	
12 ft	ultimate	1.01	1.02		
	functi.	0.94	0.96		
	damage	0.89	0.94		

For each configuration, use the lowest value from the ultimate limit state, functionality limit state and damaged condition.

The values shown in the tables for the damaged limit states shall be increased by 0.10 for bridges provided with a distributed set of diaphragms.

A minimum value of 0.80 shall be used.

A maximum value of 1.20 shall be used.

details, particularly the web depth to thickness ratio, some noncompact sections may be able to sustain some level of permanent rotations before they shed their loads. These sections were not analyzed as part of this study. However, the engineer may analyze such situations using the step-by-step procedure proposed in Section 2.7. In general, assuming that their members do not provide any level of ductility, bridges with noncompact sections in negative bending should be penalized by applying the minimum system factor $\phi_s = 0.80$.

This observation indicates that continuous bridges with noncompact sections in the negative bending regions have lower levels of redundancy than simple-span bridges. These lower levels occur because if the bridge is designed according to current practice, then the negative bending region is expected to carry a substantial moment. Hence, when a noncompact section in negative bending reaches its maximum moment capacity under the effect of a heavy load, it will "shed" *all* the moment that it carries to the adjacent sections in negative bending as well as the adjacent sections in positive bending. This will likely result in producing a failure in another noncompact section, which will also try to shed its load. Eventually, the load will have to be carried by the sections in positive bending. Because the sections in positive bending were designed for lower bending moments than simple-span bridges and are normally already quite heavily loaded, then they will be unable to carry the new moment that they receive from the negative bending regions (this moment is usually quite high for continuous beams). The sections in positive bending will try to send the new load laterally to an adjacent member, which is also carrying a significant moment. In that manner a quick unzipping of the structure will occur. This mode of failure is quite different from the case where the sections in negative bending are compact because compact sections will retain the moment that they are carrying and will shed only the "excess" moment.

Bridges with noncompact sections in the negative bending regions are different from simple-span bridges because simple-span bridges are designed to carry all the moment in positive bending. There is no transfer of load from negative regions to positive regions. However, if the positive bending regions of continuous bridges with noncompact sections in negative bending are overdesigned, then they will be able to carry the load that is "sent" to them by the failing noncompact negative sections. However, if the bridges are so overde-

TABLE 5 System factors for redundancy of continuous, steel I-beam bridges

Spacing		4 Beams	6 Beams	8 Beams	10 Beams
4ft	ultimate	0.83	1.03	1.04	1.03
	functi.	0.95	1.11	1.13	1.13
	damage	1.31	1.47	1.48	1.48
6 ft	ultimate	1.03	1.07	1.06	1.06
	functi.	1.11	1.15	1.15	1.15
	damage	1.25	1.32	1.32	1.32
8 ft	ultimate	1.06	1.07	1.07	1.07
	functi.	1.13	1.15	1.15	1.15
	damage	1.19	1.22	1.22	1.22
10 ft	ultimate	1.06	1.07	1.07	
	functi.	1.15	1.16	1.16	
	damage	1.09	1.10	1.10	
12 ft	ultimate	1.04	1.05		
	functi.	1.14	1.15		
	damage	0.99	0.99		

For each configuration, use the lowest value from the ultimate limit state, functionality limit state and damaged condition.

The values shown in the tables for the damaged limit states shall be increased by 0.10 for bridges provided with a distributed set of diaphragms.

A minimum value of 0.80 shall be used.

A maximum value of 1.20 shall be used.

signed, then "they have already been penalized" as this study's results suggest.

Table 6 is applicable for continuous, prestressed concrete bridges where prestressing is acting in the positive bending regions and regular reinforcement in the negative bending regions. Continuity of the prestressed concrete bridges applies only to the superimposed dead load and the live load only. Thus, Table 6 is not applicable to bridges designed for continuous action for all (permanent and superimposed) dead and live loads.

The results in Tables 3 through 6 are shown as a function of the number of beams and the beam spacings. The tables give three system factor values, one for each limit state (ultimate, functionality, and damaged condition). The final system factor to be used for a bridge with a given number of beams and beam spacings is the minimum of the three system factors given for that particular configuration. In addition, the maximum value that can be used is 1.20 and the minimum value is 0.8. All three system factors, as well as minimum and maximum values, are listed separately in the tables, rather than just the governing value, to provide the engineer with the option of deciding which limit states are applicable to the particular bridge being analyzed. For example, with the approval of the bridge owners, the bridge engineers may not choose to consider the damaged limit state if they are certain that the particular bridge being analyzed is protected from possible damage and loss of capacity to any of the main load-carrying members.

2.8.2 Effect of Span Lengths

The values shown in Tables 3 through 6 are the average system factors for the different span lengths analyzed. By averaging over all the span lengths considered, the effects of the moment capacity over dead load, R/D ratio, and of the span length on the results are smoothed out. However, this assumes that the R/D ratio falls within the range observed in current LFD and LRFD designs. The tables are applicable for bridges with spans that range from 45 to 150 ft in length. For longer span bridges, the engineer should use the step-by-step procedure described in Section 2.7.

The ϕ_s values in Tables 4 and 6 are the average values for all the span lengths considered. These values also correspond to the values obtained for the 100-ft bridges. Appendix B

TABLE 6 System factors for redundancy of continuous, prestressed concrete I-beam bridges

Spacing		4 Beams	6 Beams	8 Beams	10 Beams
4ft	ultimate	0.93	1.08	1.10	1.10
	functi.	0.95	1.04	1.05	1.05
	damage	1.20	1.35	1.35	1.35
6 ft	ultimate	1.04	1.08	1.08	1.08
	functi.	1.00	1.03	1.04	1.04
	damage	1.05	1.10	1.10	1.10
8 ft	ultimate	1.04	1.05	1.05	1.05
	functi.	1.00	1.02	1.02	1.03
	damage	0.92	0.95	0.95	0.95
10 ft	ultimate	1.02	1.03	1.03	
	functi.	1.00	1.02	1.02	
	damage	0.80	0.80	0.80	
12 ft	ultimate	1.00	1.01		
	functi.	1.00	1.02		
	damage	0.70	0.70		

For each configuration, use the lowest value from the ultimate limit state, functionality limit state and damaged condition.

The values shown in the tables for the damaged limit states shall be increased by 0.10 for bridges provided with a distributed set of diaphragms.

A minimum value of 0.80 shall be used.

A maximum value of 1.20 shall be used.

shows that because of the effect of the dead load, the results of the system factors for the damaged condition of prestressed concrete bridges are more sensitive to the span length than the other limit states. Therefore, to correct the tables to account for the effect of the span length on the results of the damaged condition of prestressed concrete bridges, it is recommended that the values in Tables 4 and 6 be increased by a factor equal to 0.04 for every 10-ft decrease in span length. Similarly, the factors should be decreased by 0.04 for every 10-ft increase in span length. For example, if the ϕ_s value given in the table for a six-beam, prestressed concrete bridge with 6-ft spacings is 1.25 for the damaged condition, then the ϕ_s value for the damaged condition of a 150-ft bridge should be lower than 1.25 by 0.20 (0.04×5) (i.e., the system factor for the damaged limit state is 1.05).

2.8.3 Effect of Number of Beams and Beam Spacings

The results of Tables 3 through 6 show that, for a given number of beams, the ϕ_s factors of the ultimate limit state have a tendency to increase as the beam spacing is increased from 4 ft to 8 ft. However, the factor decreases as the beam spacing is increased beyond 8 ft. This trend is explained by the fact that, for narrow bridges, all the beams are almost equally loaded, and there is no reserve strength available. If one beam fails, then all the beams will quickly follow suit. However, as the beam spacing is increased, the load distribution is uneven, and the least loaded members will pick up the load as the most heavily loaded member goes into the inelastic range. As the spacing becomes very large, despite the increased slab resistance, the capacity of the slab to transfer the load decreases, and damage to the members under the applied load occurs before a complete transfer to the other members is possible.

For a given beam spacing, the system factor ϕ_s increases as the number of beams in the system is increased. However, ϕ_s quickly reaches an asymptotic limit as the number of beams increases. The number of beams at which the asymptotic limit is reached depends on the beam spacings. Wider spacings produce lower numbers of beams for which the limiting ϕ_s value is reached.

Tables 3 through 6 were developed for bridges with four or more beams under two lanes of traffic. However, when the beam spacing is large (e.g., 12 ft), the effect of the number of beams on the ϕ_s factors becomes negligible (one exception is the damaged limit state of simple-span bridges). Because two- and three-beam bridges normally have a minimum of 12-ft spacings, then the factors obtained for the four-beam bridge with 12-ft spacings may be used for all the two- and three-beam continuous bridges and for the simple-span, two- and three-beam bridges at the ultimate and functionality limit states. For the damaged condition of simple-span bridges, the effect of the number of beams is still critical even for wide spacings. Therefore, it is proposed to use the minimum ϕ_s of 0.8 for the damaged condition of simple-span, two- and three-beam bridges.

2.8.4 Ductility Requirements

Comparing Tables 3 and 5 and Tables 4 and 6, it is noticed that span continuity would improve the redundancy of bridges especially for the damaged limit state only if the negative bending regions are ductile (compact for steel bridges) and provide high levels of reserve strength, as is the case when they are designed to carry the continuous dead and live loads. For the cases where the negative bending region is brittle (e.g., noncompact sections), the presence of continuity lowers the level of redundancy. This is because as the bridge members begin to reach their capacity limits, they tend to shed their loads transversely and longitudinally to the adjacent members and spans. To secure this transfer of load, high levels of member ductility are required.

To secure sufficient levels of redundancy, the ductility levels should be such that the bridge would be capable of withstanding a displacement at least equal to span length/100 before its ultimate capacity is attained. The span length/100 is used to indicate that, as a minimum, the ultimate limit state should not be reached before the functionality limit state. Steel members (whether composite or noncomposite) in positive bending normally provide these levels of ductility. Steel members in negative bending normally require compact sections to provide such ductility levels. For concrete members, such levels of ductility are assumed to be reached when the nominal maximum plastic hinge rotation is on the order of 1/50. This 1/50 value is obtained by assuming that the span length/100 displacement is reached when a mechanism develops by forming hinges at the midpoint and at the supports of the span while the remaining sections of the span are rigid. An approximate formula often used to calculate the maximum plastic hinge rotation of concrete members is given as

$$\theta_p = \frac{\epsilon_u d}{c} \tag{19}$$

where θ_p is the maximum plastic hinge rotation that a concrete member can sustain, d is the effective depth of the section, ϵ_u is the strain at crushing, and c is the distance from the top of the compression side of the section to the neutral axis when the ultimate member capacity is reached. Because of beam flexibility, the deflection of span length/100 is normally reached at plastic hinge rotations less than 1/50. Also, actual concrete crushing is known to occur at values of strain higher than the nominal (i.e., design) values. Hence, ensuring that the concrete beam can sustain plastic hinge rotations of 1/50 by using an approximate equation (e.g., Equation 19) and using nominal concrete properties will ensure that the redundancy levels implied in Tables 4 and 6 are reached.

2.8.5 Effect of Skew Angle

The sensitivity analysis performed in Appendixes B and C shows that the system factors given in the table for the ultimate limit state and functionality are valid for bridges with skews less than 45 deg. However, bridges with 45 deg skews and low aspect ratios (span length/bridge width) would require higher system factors for the damaged limit state. Therefore, it is proposed to limit the use of the system factor tables to bridges with skews smaller than 40 deg.

2.8.6 Box-Girder Bridges

Appendix D analyzed prestressed concrete, spread box beam, and multibox beam bridges. The results obtained indicate that spread box-girder bridges can be analyzed as I-beam bridges where each web of the box represents one I-beam. On the other hand, multibox beam bridges with adjacent boxes can be modeled as I-beam bridges where the equivalent I-beams are located at the center of each box. This modeling assumes that, for the damaged scenario, the boxes lose their torsional stiffness as well as their bending capacity (for spread boxes, the bending capacity of only one web is lost).

2.8.7 Effect of Diaphragms

A major factor influencing the results obtained in this study is the effect of the diaphragms on the system factors of damaged bridges. The tables were developed for bridges with no diaphragms. Results of the sensitivity analysis performed in Appendixes B and C (e.g., Table B.10) show that the presence of diaphragms did not significantly alter the results of the intact bridges (i.e., for the ultimate and functionality limit states). On the other hand, the system factor for the damaged limit state would increase by 0.10 for bridges with a diaphragm at the midspan where the diaphragm has a strength and stiffness about equal to that of one longitudinal girder. Bridges with diaphragms only at the ends of the span did not show any significant change in their system factors because diaphragms normally have strengths less than those of the longitudinal members. Therefore, it is recommended to increase the values shown in the tables for the damaged limit states by 0.10 for bridges provided with a distributed set of diaphragms.

The LRFD specifications do not require the use of diaphragms in bridge construction. On the other hand, the standard specifications require diaphragms at 25-ft intervals. To be consistent with current practice, it is proposed to increment the damaged system factors by 0.1 if the diaphragms follow the requirements of the standard specifications (i.e., placed at 25-ft intervals). If the diaphragms are placed only at the supports, then no changes in the system factors should be performed. To be useful, the diaphragms should be securely tied to the longitudinal beams and must have high levels of ductility to remain effective throughout the loading process. The 0.1 increase is applied before the minimum 0.8 or the maximum 1.2 values are applied.

2.8.8 Number of Traffic Lanes

Tables 3 through 6 were developed for bridges subjected to two lanes of traffic. It is evident that the level of redundancy decreases as the number of loaded lanes increases. This is because as more of a bridge's width is loaded, most of the bridge's members will be subjected to high levels of load such that if one member fails, the other members (being heavily loaded) would not have enough reserve strength to pick up the load of the failed member. The probability of having several lanes loaded simultaneously to their capacity is low. No data on such probabilities are available because available studies on bridge load modeling dealt exclusively with one- and two-lane bridges (*3,5*). The LRFD code (*2*) proposes multipresence factors "m" that vary between 1.20 for one-lane bridges, 1.00 for two-lane bridges, 0.85 for three-lane bridges, and 0.65 for bridges loaded by more than three lanes. These multipresence factors are applied on the maximum load effect obtained in one member when all the bridge lanes are loaded to account for the low probability of such a situation. Because there are no sufficient data at this point to study the relationship between the performance and redundancy of bridge systems and multilane bridge loadings, it is recommended to use Tables 3 through 6 for all bridges with two or more lanes of traffic.

On the other hand, the analysis of one-lane versus two-lane bridges in Appendixes B and C showed large variations in the effect of the number of loaded lanes depending on the number of beams and the beam spacings. Therefore, to accommodate narrow bridges that are known to carry only one lane of traffic, it is proposed to increment the system factors obtained for narrow bridges (e.g., four beams at 4 ft, four beams at 6 ft, and six beams at 4 ft) by 0.1.

2.8.9 General Comments

Tables 3 through 6 can be used during the routine design and evaluation of common-type bridges by including the appropriate system factor ϕ_s in Equation 18. For bridges whose configurations are not covered in the tables, the design and/or evaluation engineer should use the step-by-step procedure outlined in Section 2.7. The use of the tables will be illustrated in Chapter 3.

2.9 LOAD RATING OF EXISTING BRIDGES

As shown, the system and redundancy factors are intended to be used with the design of new bridges or the load capacity evaluation of existing bridges by modifying the strengths of the members using Equation 17 or 18. It is often difficult to change the member capacities of existing bridges as this may require costly rehabilitations. Therefore, instead of changing the member capacities, the evaluating engineer may simply choose to account for bridge redundancy and system safety by changing the load rating of a given bridge.

According to the 1996 AASHTO specifications (*1*), rating an existing bridge is normally performed by calculating a rating factor (R.F.) as shown:

$$\phi R_{exist} = \gamma_d D_n + \gamma_l R.F.\, L_n(1 + I) \tag{20}$$

where R.F. is the rating factor, ϕ is the resistance factor, γ_d is the dead load factor, γ_l is the live load factor, R_{exist} is the existing member capacity, D_n is the nominal or design dead load, and L_n (1 + I) is the nominal or design live load including the dynamic impact factor (I). Equation 20 has a general format that can be used for any AASHTO criteria. For example, for WSD criteria, γ_d and γ_l are given as 1.0, and ϕ is equal to 1/0.55 for inventory ratings and 1/0.75 for operating ratings.

The design and evaluation codes normally require that member resistances satisfy an equation of the form

$$\phi R_{req} = \gamma_d D_n + \gamma_l L_n(1 + I) \tag{21}$$

where R_{req} is the required member capacity and ϕ, γ_d, γ_l, D_n, and L_n (1 + I) are the same values used in Equation 20. By comparing Equations 20 and 21, R.F. can be expressed as a function of the available capacity and the required capacity, such that

$$R.F. = 1.0 + \frac{\phi(R_{exist} - R_{req})}{\gamma_l L_n(1 + I)} \tag{22}$$

Assuming that an existing bridge has a capacity R = R_{exist}, a dead load D = D_n, and a system factor ϕ_{red}, then, as shown in Equation 17, the updated required capacity is R′, such that

$$R' - D' = \frac{R - D}{\phi_{red}} \tag{23}$$

where R′ is the updated required member resistance (after application of the system factor), D′ is the updated dead load moment, R and D are the original or existing values of the resistance and the dead load effect. In many cases, changing the member capacity from R to R′ does not produce a change in the applied dead load. Thus, the updated dead load D′ is generally equal to D = D_n. Accounting for bridge redundancy, the required member capacity is R_{req} = R′. Plugging R′ into Equation 22 gives

$$R.F. = 1.0 + \frac{\phi(R_{exist} - R')}{\gamma_l L_n (1 + I)} \tag{24}$$

The R.F. is less than 1.0 for nonredundant bridges designed to satisfy current AASHTO criteria and is greater than 1.0 for redundant bridges designed to satisfy current AASHTO criteria.

The steps to calculate the R.F. of an existing bridge can be summarized as follows:

- Determine the required member capacity for a bridge member R′, either using Equation 17 if the bridge configuration is not covered by Tables 3 through 6, or using Equation 18 if Tables 3 through 6 cover the bridge configuration being considered; and
- Use Equation 24 to find the R.F.

Section 2.10 proposes a set of specifications and commentaries that can be included in future editions of the AASHTO *Standard Specifications for Highway Bridges*. The proposed set summarizes the discussion presented in Sections 2.7, 2.8, and 2.9 of this chapter.

2.10 SPECIFICATIONS

REDUNDANCY REQUIREMENTS

A. General

Bridges shall be designed to satisfy minimum levels of superstructure redundancy such that the failure of one member would not lead to catastrophic collapse. Two methods are proposed to account for the level of redundancy in the design and evaluation of bridge superstructures. The system factor approach and the incremental analysis approach. The system factors are provided for typical bridge configurations with parallel members of equal capacity. The incremental analysis approach is recommended for all other configurations.

A.1 System Factors

System factors are included in the design equations for the strength limit state to reflect the level of redundancy of typical bridge configurations. Such that:

$$\phi_s \, \phi \, R'_n = \sum \gamma_i Q_i \qquad \text{(A.1)}$$

where:

ϕ_s = system factor; a statistically based multiplier relating to system redundancy.
R'_n=modified nominal strength accounting for the system's redundancy.
R_n = nominal strength obtained if ϕ_s =1.
Q_i = force effect
γ_i = load factor; a statistically based multiplier applied to force effect.

Commentary

Redundancy is defined as the capability of a bridge to continue to carry loads after the damage or the failure of one of its members. Member failure can be either ductile or brittle. It could be caused by the application of large live loads or the sudden loss of one element due to brittle fracture or an accident.

The system safety factor approach is described in section A.1. Section A.2 describes the incremental analysis approach.

The system factor in equation A.1 replaces the η load modifier factor of the 1993 AASHTO LRFD Specifications.

The system factor of equation A.1 and the redundancy factors of equation A.8 are calibrated such that bridges that do not provide sufficient levels of redundancy are penalized by requiring their members to have higher strengths than normally required for members of bridges with redundant configurations.

Existing bridges whose strengths cannot be changed should have lower rating factors. The rating factors shall be calculated as shown in section A.3 Equation A.9.

System factors for typical configurations of simple span and continuous steel and prestressed concrete I-beam bridges shall be taken from Tables 3 through 6. Three values are given for each bridge configuration. These correspond to the ultimate limit state, functionality limit state and damaged condition. Normally, the value to be used in equation (A.1) is the minimum value of the three given. In addition, an upper limit of 1.20 and a lower limit of 0.80 shall be applied.

Bridge owners may drop the functionality or the damaged limit states if either one is determined as not applicable for the particular bridge being analyzed. For example, if all the main load carrying bridge members are protected from potential damage, the damage limit state may be ignored. Similarly, if the bridge is deemed not to be of operational importance, the functionality limit state may be dropped.

For bridges with configurations not shown in the tables, ϕ_s may be determined by interpolation and extrapolation. Otherwise, use the incremental analysis procedure of section A.2.

Span length

The factors given in the tables are for bridges with 100 ft spans. They are applicable to all other span lengths with the following exception: System factors for the damaged limit state of simple span and continuous prestressed concrete bridges with spans different

Three system limit states are defined in the context of bridge redundancy. These are:

Ultimate Limit State defined as the formation of a collapse mechanism or the occurrence of major damage rendering the bridge unfit for use.

Functionality limit state defined as the capability of the system to resist large levels of deformations defined as span length/100. This deflection limit is chosen herein because it is judged to correspond to such large permanent deformations that would render the bridge unfit for traffic.

Damaged limit state defined as the capability of the system to still carry some live load after the loss of one member.

The flexibility in choosing the limit states is meant to replace the operational importance factor, η_i, of the 1993 LRFD Specifications.

Operational importance should be based on social/survival and/or security/defense requirements as specified in the LRFD Specifications.

The system factor tables are calibrated to satisfy system reliability indices equivalent to those obtained from bridges known to have redundant configurations.

than 100 ft long shall be modified by increasing the system factor by 0.04 for every 10 ft decrease in span length below 100 ft. Similarly, the factors of the tables should be decreased by 0.04 for every 10 ft increase in length above 100 ft. The 1.2 and 0.8 limits are still applicable.

Ductility requirements

All steel members in positive bending and compact steel sections in negative bending possess sufficient levels of ductility to produce the required levels of redundancy.

Bridges with noncompact steel sections in negative bending are not redundant and a system factor of 0.8 shall be used.

Prestressed concrete members should be detailed to withstand a plastic hinge rotation greater than 1/50.

To secure sufficient levels of redundancy, bridge members must be capable of withstanding plastic hinge rotations greater than 1/50.

The 1/50 rotation limit is imposed to ensure that the bridge's ultimate limit state implied in the tables is reached.

An approximate formula proposed to check the maximum plastic hinge rotation of prestressed concrete members is given as:

$$\theta_p = \frac{\epsilon_u d}{c}$$

where θ_p is the maximum plastic hinge rotation that a concrete member can sustain, d is the effective depth of the section, ε_u is the strain at crushing and c is the distance from the top of the compression side of the section to the neutral axis when the ultimate member capacity is reached.

Effect of skew angle

Tables 3 through 6 are valid for bridges with skews smaller than 40 degrees.

Box Girder bridges

System factors for spread box girder bridges shall be taken from Tables 3 through 6 by modelling each web of a box as an equivalent I-beam.

System factors for multi-box girder bridges shall be taken from Tables 3 through 6 by modelling every box by an equivalent I-beam placed at the center line of the box.

Effect of diaphragms

Bridges with a set of diaphragms at 25 ft intervals shall have the system factors shown in the tables for the damaged limit state incremented by 0.10. The 0.8 and 1.2 minimum value are still applicable.

The 0.1 increment is applied only for the damaged limit state.

To be consistent with current practice as required by the AASHTO Standard Specifications, it is recommended to apply the 0.1 increment only if the diaphragms are spaced at less than 25 ft intervals.

Traffic lanes

Tables 3- 6 are applicable for all bridges with two or more lanes. The system factors given in the tables shall be incremented by 0.1 for narrow one-lane bridges.

Examples of narrow bridges include bridges with 4 beams at 4 ft spacings, four-beam bridges with 6 ft spacings and bridges with 6 beams at 4 ft spacings.

A.2 Incremental Analysis Procedure.

To consider the redundancy of bridges with configurations that are not covered in the tables an incremental analysis shall be performed. The analysis requires the use of a computer program capable of modeling the linear and nonlinear behavior of bridge systems. The steps to be followed are:

Step 1. Use a system ϕ_s=1.0 to find the required member capacity, R_n, for the bridge members using equation A.1.

Step 2. Develop a structural model of the bridge to be used with a finite element package that allows static nonlinear analysis. In the model, use the best estimate of the nonlinear material properties of the structural members without applying any safety factors or resistance factors. Use a best estimate for the material properties. Apply a best estimate of the unfactored dead load.

Step 3. Identify lateral and longitudinal loading positions on the structure for the AASHTO design trucks to produce the most critical loading effect. Do not include impact factors.

Step 4. Apply the loads to the structure and perform a <u>linear</u> elastic analysis to calculate $L_{HS\,20}$ which gives the effect of the AASHTO design trucks on the most critical member. Using equation A.2, calculate $LF_{1\,req.}$.

This section presents a direct method to determine the redundancy level of a bridge system using a detailed nonlinear load-deflection analysis. The procedure can be used in conjunction with any member checking criteria including AASHTO's WSD, LFD or LRFD including either inventory or operating ratings. The basic procedure has been calibrated on the basis of providing adequate redundancy levels for two-lane bridges.

The purpose of this analysis is to determine the ultimate limit capacity, the capacity of the bridge to resist the functionality limit and the capacity of the bridge to sustain live loads after a loss of a main member. These system limit states will be compared to the load capacity of the bridge components to obtain a measure of the system's redundancy.

Load and resistance factors are not used at this stage since they will be considered when checking the final results.

No dynamic impact factors are required during the load-deflection analysis.

The most critical member is the main member that will fail first.

The required member load factor, $LF_{1\,req.}$ is defined as:

$$LF_{1\,req.} = \frac{R_n - D_n}{L_{HS20}} \qquad A.2$$

where R_n is the required member capacity obtained from equation (A.1), using ϕ_s=1.0. D_n is the dead load effect on the most critically loaded member and $L_{HS\,20}$ is the effect of the AASHTO design vehicle load on the most critical member.

Step 5. In the computer model, increment the loads of the AASHTO design trucks until the first member reaches its limiting capacity. Determine the load factor LF_1 by which the original design trucks are scaled for the first member failure to occur. Then, using equation A.3, calculate the member reserve ratio r_1.

$$r_1 = \frac{LF_1}{LF_{1\,req.}} \qquad A.3$$

Step 6. Using a nonlinear structural model and nonlinear material properties, increment the applied design truck loads until the maximum vertical deflection of a primary member reaches a deflection equal to the span length divided by 100 (span/100). Determine the load factor LF_f by which the original design trucks are scaled to achieve the span/100 displacement level.

If the ratio $R_f = LF_f/LF_1$, is higher than 1.1, then the bridge has a sufficient level of redundancy to satisfy the functionality limit state.

$L_{HS\,20}$ is calculated from a linear elastic analysis of the bridge. The HS-20 truck is used as a representative model. The same steps can be performed using HS-25 truck or any other appropriate truck model.

As in the case of $LF_{1\,req.}$, the calculation of LF_1 is performed with the AASHTO design trucks without including the impact factor.

r_1 provides a measure of the adequacy of the actual member capacity represented by LF_1 to that required by the AASHTO specifications. Bridge members that are designed to exactly match the AASHTO specifications will produce a member reserve ratio=1.0. Members that are overdesigned will produce r_1 values higher than 1.0. It should be noted that this member evaluation procedure can be used with any bridge design criteria including WSD, LFD and LRFD or bridge evaluation procedures including operating and inventory rating levels.

Calculate the redundancy ratio for functionality r_f:

$$r_f = \frac{R_f}{1.10} \qquad A.4$$

Step 7. Increment further the load until the ultimate limit state is reached; this is defined as the load at which a mechanism forms. The load factor calculated in this step is LF_u. If $R_u = LF_u/LF_1$ is larger than 1.30, then the bridge has a sufficient level of redundancy to satisfy the ultimate limit state. Calculate the redundancy ratio r_u:

$$r_u = \frac{R_u}{1.30} \qquad A.5$$

Step 8. Determine the appropriate damage scenario to be checked by identifying members susceptible to damage whose failure might be critical to the structural integrity of the bridge.

Step 9. Remove one of the members identified in step 8 from the structural model and repeat the nonlinear analysis. Determine the ultimate load factor of the damaged bridge LF_d. If the ratio $R_d = LF_d/LF_1$ exceeds 0.50, the bridge provides a sufficient level of damage redundancy. Calculate the redundancy ratio for damaged conditions:

$$r_d = \frac{R_d}{0.50} \qquad A.6$$

The damage scenarios should be specified in consultation with the bridge owner. Members susceptible to damage could be: a) Members that can be damaged by an accidental collision by vehicles, ships or debris. b) Prestressed concrete members that might loose prestressing due to fatigue of cables. c) Steel members that are prone to fatigue or are fracture critical.

Step 10. Place the member removed in step 9 back into the model and remove another critical member. Repeat step 9 until all the critical members identified in step 8 are checked.

Step 11. If necessary, repeat steps 3 through 10 to cover all critical load patterns. The final values used for R_u, R_f, and R_d are the minimum values obtained from checking different loading patterns.

Step 12. If all the redundancy ratios r_u, r_f and r_d obtained from your analysis are larger than 1.0, the bridge has a sufficient level of redundancy. If any redundancy ratio is smaller than 1.0, the bridge does not have a sufficient level of redundancy and corrective measures should be undertaken to improve bridge safety.

Redundancy Factors

The redundancy factor ϕ_{red} is defined as:

$$\phi_{red} = \min(r_1 r_u,\ r_1 r_f,\ r_1 r_d)$$

$$0.8\, r_1 < \phi_{red} < 1.2\, r_1 \qquad A.7$$

where r_1 is the member reserve ratio defined in equation A.3, r_u is the redundancy ratio for the ultimate limit state defined in equation A.5, r_f is the redundancy ratio for the functionality limit state defined in equation A.4 and r_d is the redundancy ratio for the damaged condition defined in equation A.6.

The procedure is applicable for all bridge configurations including simple span and continuous bridges. The procedure is also valid for all material types.

One-lane and multi-lane bridges are analyzed using the same step-by-step procedure outlined herein with the appropriate number of AASHTO design trucks. Since the redundancy approach uses the normalized ratios of LF_u/LF_1, LF_f/LF_1 and LF_d/LF_1 the same procedure can be used for one-lane and multi-lane bridges without requiring any corrections to the proposed procedure.

The analysis can also be performed for any other truck load model using the same procedure.

Corrective measures to improve bridge redundancy and/or overall bridge safety may include a change in the bridge topology, the strengthening of bridge members or decreasing the rating of the bridge.

To ensure that a minimum level of member safety is maintained, it is herein recommended that maximum r_u, r_f and r_d values of 1.20 be used as upper limits (i.e. use 1.20 as a maximum limit on ϕ_{red} before applying r_1). A minimum value of 0.8 is recommended so that the final results remain reasonably close to those obtained with current practice.

If ϕ_{red} is less than 1.0, it indicates that the bridge under consideration has an inadequate level of system safety and redundancy. A redundancy factor greater than one indicates that the level of system safety is adequate.

To improve the redundancy of a bridge, the geometric configuration should be changed by adding members. If this cannot be achieved, nonredundant bridges are penalized by requiring their members to provide higher safety levels than those of similar bridges with redundant configurations.

The member strengths of nonredundant bridges should be improved by increasing their member reserve capacities (R_n-D_n) by a factor equal to $1/\phi_{red}$ as shown in equation A.8.

$$R'_n - D'_n = \frac{R_n - D_n}{\phi_{red}} \qquad A.8$$

where R'_n is the resistance required to satisfy the redundancy criteria proposed in this study. D'_n is the updated dead load corresponding to the member with a resistance R_n. R_n is the original resistance in the member, D_n is the original dead load in the member, and ϕ_{red} is the redundancy factor.

As presented in equation (A.8), the ϕ_{red} factor is a penalty-reward function whereby bridges with nonredundant configurations would be required to have higher member capacities than similar bridges' with redundant configurations. By strengthening the members, an overall satisfaction of the system reliability targets is achieved. On the other hand, redundant configurations will be rewarded by allowing their members to have lower capacities.

If the members of an existing bridge cannot be strengthened, then the bridge rating should be lowered by applying a rating factor as shown in section A.3.

In principle, the same redundancy factor, ϕ_{red}, should be applied to all the members of the bridge system. In reality, some members may contribute less than other members toward the overall system capacity and using the same ϕ_{red} factor for all the members may be inefficient. To be more efficient, the redundancy factor ϕ_{red} may be applied to the most critical member(s) only and the full analysis described repeated until the system redundancy requirements are satisfied.

It should be noted that, applying a redundancy factor ϕ_{red} will improve the bridge members' strengths represented by LF_1 and will also improve system strength expressed in terms of LF_u, LF_f and LF_d. Thus, the system ratios R_u, R_f and R_d will practically remain unchanged and a nonredundant bridge will remain nonredundant. However, by applying a redundancy factor ϕ_{red} less than 1.0, the reliability index for one member as well as the system reliability indices will be increased to the desired target level. Thus, nonredundant designs are penalized by requiring higher member safety levels than similar bridges with redundant configurations.

A.3 Load Rating

Bridges that do not satisfy the redundancy requirements set in these specifications must have their configurations changed or should be penalized by applying the system and the redundancy factors calculated as described in sections A.1 and A.2. If neither action is possible, then the load rating of such bridges should be lowered. This can be achieved by calculating a rating factor R.F. as follows:

Step 1. Determine the required member capacity for a bridge member, R'_n using either the incremental analysis of section A.2 and equation A.8 or the system factor tables with equation A.1

Step 2. Use equation A.9 to find the rating factor R.F.

$$R.F. = 1.0 + \frac{\phi \, (R_{exist} - R'_n)}{\gamma_l \, L_n (1 + I)} \qquad A.9$$

where R_{exist} is the strength capacity of the existing member. L_n is the design live load effect, I is the dynamic impact factor, γ_l is the live load factor and ϕ is the member resistance factor. R_n is the required member resistance accounting for bridge redundancy.

The rating factor R.F. is less than 1 if the bridge is nonredundant and R.F. is greater than 1.0 for redundant bridges designed to satisfy current AASHTO criteria.

2.11 SUMMARY

This chapter presents a methodology to consider redundancy during the design and load capacity evaluation of bridge superstructures. The proposed framework consists of tables of system factors that can be included in the design equations of the AASHTO bridge design and evaluation specifications. These system factors can be used while checking the strength of the members of bridges with parallel members of equal capacity. The tables are applicable for simple-span, multigirder prestressed concrete and steel bridges; continuous prestressed concrete and steel I-beam bridges; prestressed concrete, spread box beam bridges; and prestressed concrete, multibox beam bridges with skews less than 40 deg.

For bridges that are not covered by the tables, a direct analysis procedure is proposed to calculate the redundancy factor based on a nonlinear incremental analysis of the superstructure. The proposed approach, the direct system redundancy check, is very general and is applicable to any bridge type with any design load. Naturally, the direct analysis can be performed only if a complete description of the bridge members and their nonlinear properties is available.

The methods developed in this study consist of penalizing nonredundant designs by requiring more conservative member capacities than required by current codes. In addition, a method to change the load rating of existing bridges has been outlined. This study is concerned only with member strengths because only these strengths influence the overall system capacity. Member serviceability criteria, such as concrete cracking and fatigue, are not addressed.

A set of specifications and commentary are proposed. These specifications could be included in future editions of the AASHTO *Standard Specifications for Highway Bridges*. Chapter 3 presents several examples illustrating how the proposed methodology can be applied in practice. A comparison between the results obtained using the tables and the direct analysis is also performed. Because the methods were calibrated by averaging over several different parameters and material types, some differences between the results obtained from the two methods are expected. The results shown in Chapter 3, however, indicate that the results from these alternative approaches are reasonably close.

CHAPTER 3

APPLICATIONS

Chapter 2 provided tables of system factors that can be used to consider redundancy during the design and the load capacity evaluation of common-type bridge superstructures. In addition, Chapter 2 proposed a direct analysis method that can be used for any bridge configuration. These approaches were presented in a set of specifications and commentaries that could be included in future editions of the AASHTO *Standard Specifications for Highway Bridges*. This chapter gives examples to illustrate how the proposed methods are applied in practice. Two examples are presented: a simply supported, prestressed concrete bridge and a steel truss bridge.

3.1 SIMPLY SUPPORTED PRESTRESSED CONCRETE BRIDGE

A 100-ft, prestressed concrete bridge that satisfies AASHTO's LFD criteria with nominal HS-20 loading is to be checked for redundancy. The bridge sections' properties are different from those of the bridge used in Appendix B during the reliability calibration. The cross section of the six-girder, simply supported prestressed concrete bridge is shown in Figure 1. The girders are spaced at 8 ft center to center and the deck is 7 in. According to AASHTO's LFD criteria for this prestressed concrete bridge, the required member capacity R_{req} is calculated as

$$0.95\, R_{req} = 1.3\, D + 2.17 \times (1 + I) \times DF \times L_{wheel} \qquad (25)$$

where R_{req} is the required member capacity, D is the applied dead load, I is the AASHTO impact factor, DF is the AASHTO load distribution factor as defined in the 1996 specifications, and L_{wheel} is the live load effect of one wheel of AASHTO's HS-20 truck load.

The bridge is formed by type IV AASHTO girders that produce a dead load moment D equal to 1,970 kip-ft. I is the impact factor and, according to the 1996 AASHTO specifications, is equal to 0.22 for a 100-ft span. AASHTO's (1996) distribution factor DF is given as 1.45 for a wheel load. L_{wheel} is given as 762.5 kip-ft for the HS-20 vehicle. Equation 25 leads to a required girder moment capacity R_{req} equal to 5,780 kip-ft.

The longitudinal members, which are at 8 ft center to center, are type IV AASHTO girders with 4.80 in^2 of grade 270 prestressing steel at an effective depth of 57 in. from the top of the slab. The same effective depth is assumed for the whole span length. The effective prestressing force is equal to 726 kips. The section's concrete strength is 5,000 psi while the slab's strength is 3,000 psi. Following AASHTO's specifications, the nominal ultimate moment capacity R_n of each girder section was found to be 5,810 kip-ft. Using an exact analysis to calculate the capacity of the section, the ultimate moment capacity R was found to be 5,900 kip-ft, constituting a difference of only 1.5 percent compared to AASHTO's estimate. The exact analysis uses realistic stress-strain diagrams rather than Whitney's rectangular block to calculate the moment capacity and the moment versus rotation curve. The moment versus rotation curve for this section was obtained using the method described in Appendix B. The maximum plastic hinge rotation $\theta_{p\,max}$ determines the point at which the prestressed member fails. For the bridge members, $\theta_{p\,max}$ is equal to 0.0402 rad. The maximum plastic hinge rotation provides a measure of the member's ductility, and thus is very important to determine the maximum ultimate capacity of the bridge system. The value obtained satisfies the ductility requirements to ensure a proper transfer of the load as the bridge members go into the inelastic range. The length of the plastic hinge L_p is estimated as 57 in. (same as effective depth d).

This information is used as input data to the program NONBAN as described in Appendix F. The moment versus rotation curves for the longitudinal members are used to represent the nonlinear behavior of the composite prestressed girder sections. The behavior of the slab is also modeled as transverse beams using moment versus rotation curves similar to those of the longitudinal members. The slab is 7 in. thick and has a moment capacity of 33 kip-ft/ft. The grillage model used in conjunction with NONBAN was found to accurately account for the contribution of the slab to the transverse distribution of the load. However, such a model may not be accurate enough to monitor the deformations in the deck slab. Therefore, in this study, all the displacements and rotations are monitored in the main longitudinal members. The program NONBAN is used for convenience; however, any other program capable of performing a nonlinear incremental analysis can be used.

In a first stage, a linear elastic analysis is performed for two AASHTO HS-20 trucks without impact factor applied on the bridge as shown in Figure 1. The total moment due to

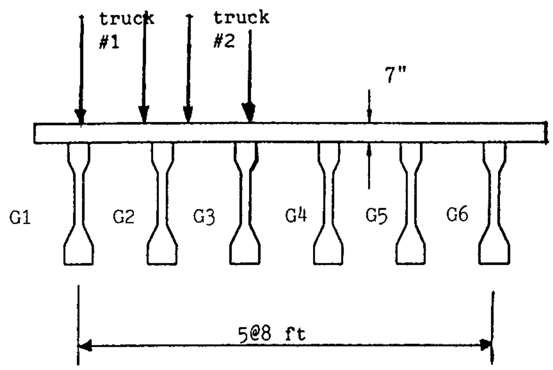


Figure 1. Layout of prestressed concrete bridge example.

the two HS-20 trucks is 3,050 kip-ft. The most heavily loaded member is the external girder G1. G1 carries a live load $L_{HS\text{-}20}$ equal to 1,148 kip-ft constituting 38 percent of the total live load. The dead load moment D was 1,970 kip-ft. Using the results of the elastic analysis, the projected load factor LF_1 that will lead to the failure of the most heavily loaded member can be calculated as

$$LF_1 = \frac{R - D}{L_{HS\text{-}20}} \tag{26}$$

where R is the actual member capacity given as 5,900 kip-ft, D is the dead load effect equal to 1,970 kip-ft, and $L_{HS\text{-}20}$ equal to 1,148 kip-ft is the effect of the two HS-20 trucks on the most heavily loaded member. Substituting into Equation 26 leads to a load factor LF_1 equal to 3.42. This indicates that, by projecting the results of a linear elastic analysis, the first member of the bridge will fail when the HS-20 truck loads are incremented by a factor equal to 3.42.

In a second stage, the AASHTO loads are incremented using a nonlinear model of the bridge structure. The maximum vertical deflections in the longitudinal girders are computed for every load step as the truck load is incremented. Figure 2 gives a plot of load factor versus displacement obtained for this bridge example. A maximum deflection of 12.0 in., constituting the span length/100, was obtained when the load factor was 3.28. This load factor is defined as LF_f. The load was further increased until one girder reached a plastic hinge rotation equal to the maximum allowable rotation of 0.0402 rad. The girder that first

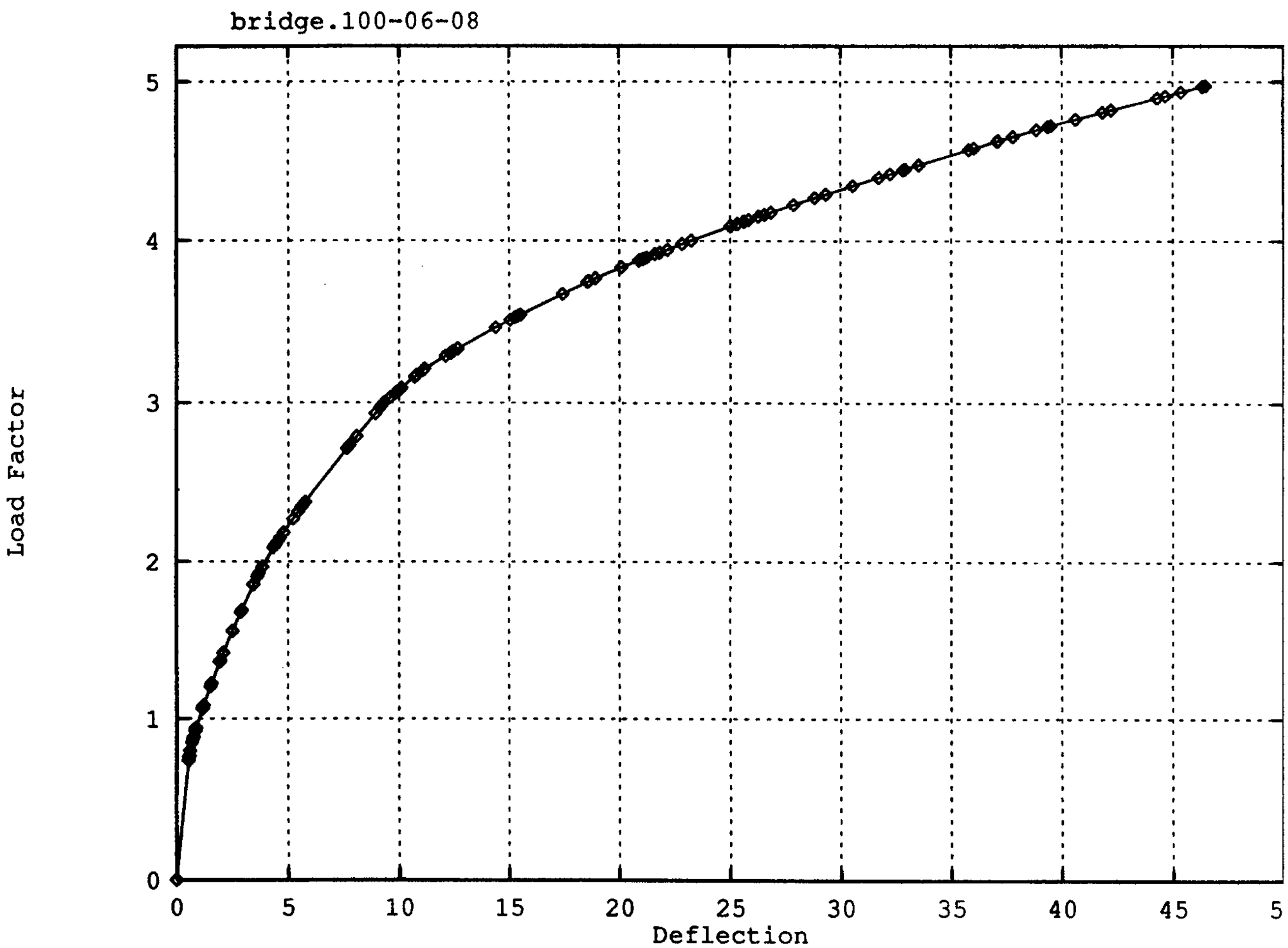


Figure 2. Load factor versus maximum displacement for prestressed concrete bridges.

reached this maximum plastic rotation was the external girder G1. At this point the bridge would undergo extensive damage, and it is assumed that the ultimate capacity is reached. The load factor at which this maximum plastic hinge rotation was reached is defined as LF_u. For this structure, LF_u was found to be equal to 4.98. This factor is about 55 percent lower than the ultimate load capacity would have been if the nonlinear analysis were continued until a collapse mechanism formed. For the case of a mechanism, the ultimate capacity would have been six times (since the bridge has six girders) the reserve capacity of one member (i.e., the resistance − dead load). This would indicate a total moment capacity of 23,580 kip-ft (6 × 5,900 − 6 × 1,970 kip-ft). A moment capacity of 23,580 kip-ft corresponds to 7.73 times two side-by-side HS-20 trucks (23,580/3,050).

The calculation of the capacity of the bridge to sustain load under damaged conditions is performed. For example, the same incremental analysis is repeated assuming that the external girder G1 was completely removed from the model. To simulate this condition, the structural model was modified by assuming that the member has zero stiffness along its total length. This condition simulates extreme damage to the girder because of severing of the prestressing strands due to fatigue failure or a collision by a truck, ship, or debris. In this example, girder G1 was chosen as the damaged girder because it was the most critical member of the intact structure. The loading pattern used in this analysis is the same as that used for the intact bridge. This assumes that the presence of the parapet, slab, and other secondary members will allow the transfer of the load to the remaining members of the bridge. The HS-20 loads applied on the damaged bridge are incremented until the plastic hinge rotation of any remaining member reaches the limiting value of 0.0402 rad. This failure occurred at a load factor LF_d equal to 2.59. In this case, the member that failed is G2.

3.1.1 Direct Redundancy Analysis

The procedure outlined in Section 2.7 is used to evaluate the redundancy of the six-girder, simply supported prestressed concrete bridge system. The analysis is performed for the six-girder bridge as originally designed with member capacities equal to 5,900 kip-ft. The load factor for the failure of the first member is calculated in Equation 26. For this prestressed bridge, LF_1 was equal to 3.42. This factor indicates that, assuming a linear elastic analysis, the first member of the bridge will fail when the HS-20 truck loads are scaled by a factor equal to 3.42.

The values of LF_u, LF_f, and LF_d, obtained from the incremental analysis of the original prestressed concrete bridge, were 4.98, 3.28, and 2.59, respectively. These factors are compared to the member factor LF_1 equal to 3.42. The system reserve ratios obtained are $R_u = LF_u/LF_1 = 1.46$, $R_f = LF_f/LF_1 = 0.96$, and $R_d = LF_d/LF_1 = 0.76$. These system ratios are compared to the required system reserve ratios $R_{u\ req}$, $R_{f\ req}$, and $R_{d\ req}$ given in Table 2; the required values are 1.30, 1.10, and 0.50, respectively. The redundancy ratios are $r_u = R_u/R_{u\ req} = 1.12$, $r_f = R_f/R_{f\ req} = 0.87$, and $r_d = R_d/R_{d\ req} = 1.52$. Because one of the redundancy ratios is less than 1.0, this bridge's geometric configuration is considered to be nonredundant.

A bridge system that is adequately redundant may still be inadequate for truck traffic if its members are inadequately designed (and vice versa). Redundancy recognizes the difference between the system strength and the member strength but not individual member strengths. Therefore, the redundancy ratios should be combined with a measure of member safety to verify that overall system safety is adequate. Checks of member safety can be performed according to any currently acceptable AASHTO criteria, including the WSD, LFD, or LRFD methods. As an example, this section uses the LRFD code to verify the adequacy of this prestressed concrete bridge.

In this part, the required member capacity is calculated according to the AASHTO LRFD criteria (*2*). The required member capacity for this prestressed concrete bridge is given as

$$1.0\, R_{req} = 1.25\, D + 1.75 \times (IM \times L_{HS\text{-}20} + L_{lane}) \tag{27}$$

The dead load moment D is given as 1,970 kip-ft. IM is the impact factor applied on the axles only and, according to the LRFD specifications (*2*), is given as 1.33 for all span lengths. $L_{HS\text{-}20}$ is the effect of the AASHTO truck on the most heavily loaded member and is given as 1,148 kip-ft from the calculations of NONBAN. It is also possible to calculate this value using the load distribution factors provided in the LRFD specifications (*2*). L_{lane} is the effect of the distributed lane load. For the 100-ft span length, the effect of the distributed lane load is equal to 640 kip-ft. Equation 27 yields a required moment capacity R_{req} equal to 6,250 kip-ft. The load factor for first member failure required to satisfy the LRFD criteria becomes

$$LF_{1\,req} = \frac{R_{req} - D}{L_{HS\text{-}20}} \tag{28}$$

Substituting $R_{req} = 6{,}250$ kip-ft, $D = 1{,}970$ kip-ft, and $L_{HS\text{-}20} = 1{,}148$ kip-ft into Equation 28 leads to an $LF_{1\ req} = 3.73$. According to the results of the linear elastic analysis, first member failure will occur in this bridge when the HS-20 trucks are scaled by a load factor LF_1 equal to 3.46. Because $LF_{1\ req} = 3.73$ is greater than $LF_1 = 3.42$, then this bridge, as currently designed, does not satisfy the LRFD requirements. This leads to a member reserve ratio $r_1 = LF_1 / LF_{1\ req}$ equal to 0.92. This means that the member capacities of this bridge produce a live load margin (R − D) lower than that required for the LRFD code by 8 percent. The redundancy factor ϕ_{red} is calculated using Equation 16 as

$$\phi_{red} = \min(r_1 r_u, r_1 r_f, r_1 r_d)$$

$$\phi_{red} = \min(0.92 \times 1.12, 0.92 \text{ x } 0.87, 0.92 \times 1.52)$$

$$\phi_{red} = \min(1.03, 0.80, 1.40) = 0.80 \qquad (29)$$

A redundancy factor ϕ_{red} equal to 0.80 indicates that the member strength must be increased as shown in Equation 17 to improve the overall system safety of this bridge. Applying Equation 17 with R = 5,900 kip-ft and D = 1,970 kip-ft, and assuming that after updating the strength of the bridge members the dead load on each member D′ is still 1,970 kip-ft, then the value of R′ calculated is

$$R' - 1{,}970 = \frac{5{,}900 - 1{,}970}{0.80} \qquad (30)$$

producing a value of R′ = 6,882 kip-ft. This means that the members must be designed for a capacity of 6,882 kip-ft to satisfy the AASHTO LRFD criteria along with the redundancy criteria specified in this study.

3.1.2 System Factor Tables

The direct analysis method required a detailed incremental analysis of the prestressed concrete bridge accounting for the nonlinear behavior of the members. This simple-span, prestressed bridge belongs to the common type of bridges for which tables of system factors have been developed in Chapter 2. Thus, the level of effort required to evaluate the redundancy of this bridge can be drastically reduced because the system factors can be simply extracted from the tables given in Chapter 2. Using the prestressed concrete tables for a simple-span bridge with six beams at 8-ft spacings and a length of 100 ft, Table 4 gives a system factor for the ultimate limit state equal to 1.07. The system factor for the serviceability limit state is 0.98. Similarly, the system factor for the damaged condition is 1.18.

Equation 18 can be used to estimate the required member capacity for the above bridge if it were to be designed taking into consideration its redundancy. Using the LRFD criteria, the resistance factor ϕ is equal to 1.00, the dead load factor γ_d is equal to 1.25, and the live load factor γ_l is equal to 1.75. D_n is the dead load moment and, for this six-girder prestressed bridge, is given as 1,970 kip-ft. L_n is the nominal live load and, for this 100-ft bridge, is given as 1.33 × 1,148 kip-ft + 640 kip-ft. (The impact factor is given as 0.33 applied on the truck load.) The system factor ϕ_s is taken as the lowest value from Table 4 for six beams at 6-ft spacings for a 100-ft span. The lowest value is equal to 0.98. Plugging these values into Equation 18 gives

$$0.98\, R_{req} = 1.25 \times 1{,}970 + 1.75 \times (1.33 \times 1{,}148 + 640) \qquad (31)$$

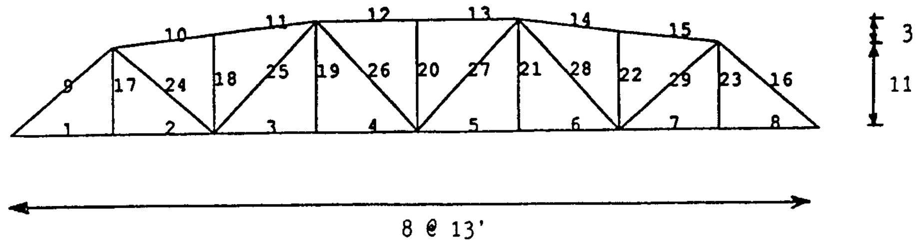
which gives a required updated member capacity equal to 6,382 kip-ft. Compare this value to the R′ value of 6,882 kip-ft obtained in Equation 30. The difference is about 7 percent.

This example illustrates that the proposed system factors can be directly used to account for bridge redundancy without performing a nonlinear analysis. This, however, requires that the bridge configuration be covered in the tables developed in Chapter 2. Some difference (approximately 7 percent) is observed between the two methods. An even larger difference would have resulted if the damaged condition had governed. The design details cause this difference. For example, the bridge designed here has a different cross-sectional configuration, dead weight, material properties, transverse slab strength, and R/D ratio than the bridges used to develop the system factor tables. Nevertheless, the difference is deemed acceptable for the purpose of penalizing this configuration, which did not satisfy the functionality criteria as determined in this study.

The results shown are consistent with current design and evaluation practice. It is well known that the design of prestressed concrete bridges is currently governed by member serviceability criteria rather than by the strength limits. Obviously when the member serviceability limits, rather than strength, govern the design of the members, the bridge is automatically "penalized."

3.2 STEEL TRUSS BRIDGE

A steel truss bridge was also analyzed to illustrate how the direct analysis can be applied in practice. The through-truss bridge has two parallel trusses similar to the one shown in Figure 3. Figure 3 gives the geometry of the bridge and a listing of the truss members along with their cross-sectional areas. The two parallel trusses are connected by cross beams and diagonals supporting a concrete deck. A three-dimensional nonlinear model of the truss is used for the structural analysis. The nonlinear behavior of the steel truss members is modeled using a bilinear stress strain curve as shown in Figure 4. The maximum strain that the steel truss members are assumed to sustain before member unloading occurs is 0.02. Although steel may reach higher strains, it is assumed that strain levels above 0.02 would create major local deformations at the connections causing unloading to occur. The same behavior is assumed to hold for both tension and compression. To simplify the calculation process, it is assumed that the compression members are braced so that buckling is avoided. The cross beams and the deck are assumed to remain in the elastic range. The incremental analysis is performed with two side-by-side HS-20 trucks loaded as shown in Figure 5. The HS-20 loads are then incremented until one member attains the limiting strain level of 0.02. The analysis performed investigates only the main truss members assuming that the connections provide enough capacity to sustain the applied loads until the strain of 0.02 is reached.



No.	Area	No.	Area	No.	Area	No.	Area	No.	Area
1	25.2	7	25.2	13	43.4	19	19.1	25	13.2
2	25.2	8	25.2	14	40.4	20	13.2	26	13.2
3	38.3	9	38.0	15	40.4	21	19.1	27	13.2
4	38.3	10	40.4	16	38.0	22	13.2	28	13.2
5	38.3	11	40.4	17	15.6	23	15.6	29	17.0
6	38.3	12	43.4	18	13.2	24	17.0		

Figure 3. Layout of steel truss bridge example.

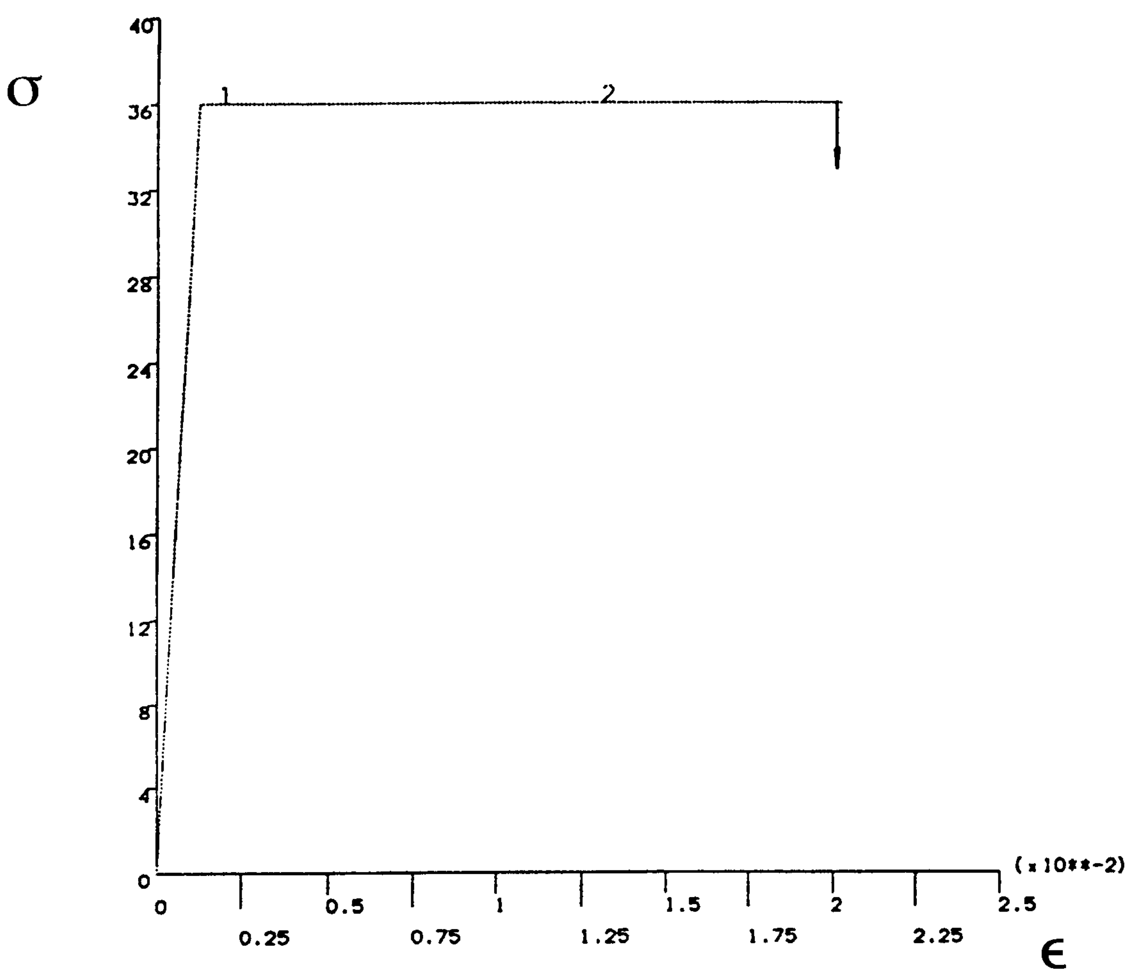


Figure 4. Stress strain diagram of steel truss members.

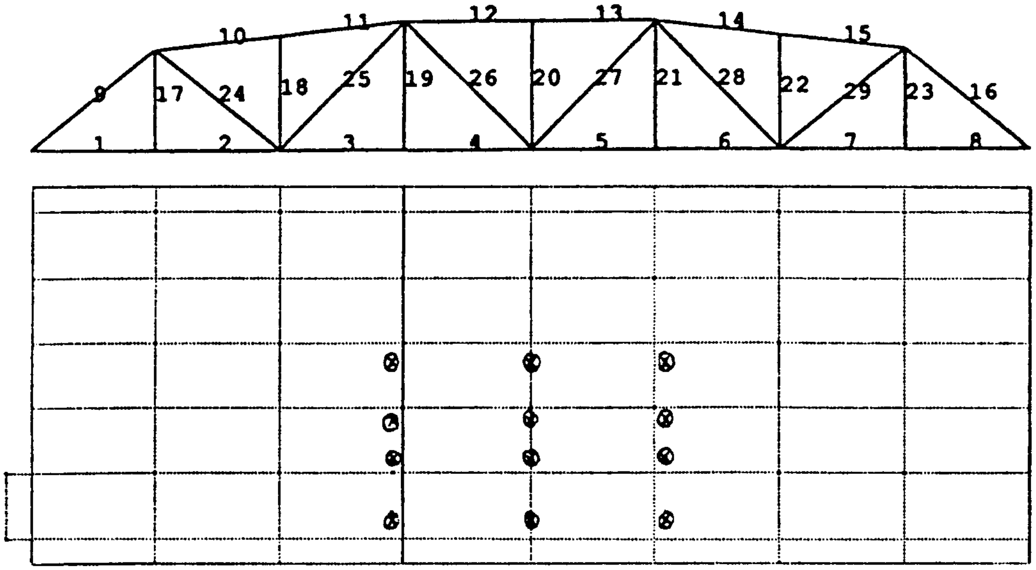


Figure 5. Loading pattern for steel truss bridge.

For every load step, the maximum vertical deflection in the truss is obtained. A plot of load factor versus maximum vertical truss displacement is plotted in Figure 6 (curve a). At a load factor of 7.6, the first member (member 29 in Figure 5) reaches its yielding point (assumed to be equal to 36 ksi). This load factor is defined as LF_1. At a load factor of 8.5, members 12 and 13 reached their yielding points. At a load factor of 9.0, the maximum vertical displacement in the truss reached a value of span length/100 (12.4 in.). This load factor is defined as LF_f. Finally, at a load factor of 9.4, the maximum strain in member 29 reached its assumed ultimate value of 0.02. At that point, unloading in member 29 is assumed to occur, and the analysis is stopped. This load factor is assumed to be the maximum capacity of the bridge before major damage occurs, and the load factor of 9.4 is defined as LF_u.

To simulate the effect of damage to one of the truss members, the incremental analysis is repeated after totally removing member 29 from the truss. This member is the most critically loaded tension member. Removing it from the structural model simulates a possible fatigue failure rendering it incapable of carrying any load. The incremental analysis is performed for the loading pattern used earlier for the intact structure. A load factor equal to 4.2 produced the yielding of member 28. At a load factor of 5.0, member 23 reached its yielding point. Finally, at a load factor of 6.2, member 23 reached a maximum strain of 0.02, indicating that unloading started and that major damage occurred rendering the bridge unfit for traffic use. This load factor for the damaged bridge with member 29 removed is defined as LF_d. Figure 6 shows the load factor versus displacement curve for this damaged condition (curve b).

Another possible damage scenario involves the damage of member 12. If member 12 is removed from the structural model and the bridge is loaded using the same loading pattern, then the load factor versus displacement curve will be as shown in Figure 6 (curve c). For this scenario, the load factor LF_d is equal to 3.23.

The final value for LF_d that should be used is the lowest value from all possible scenarios. For this bridge, LF_d is equal to 3.23.

Next, the procedure outlined in Section 2.7 to evaluate the redundancy of a bridge system is followed. This concept of system redundancy can be used in conjunction with any member checking criteria. According to the concept, the values of LF_u, LF_f, and LF_d obtained from the incremental analysis, which are 9.40, 9.0, and 3.23, respectively, are compared to the member load factor LF_1 = 7.21. The system reserve ratios obtained are $R_u = LF_u/LF_1 = 1.30$, $R_f = LF_f/LF_1 = 1.25$, and $R_d = LF_d/LF_1 = 0.45$. These system ratios are compared to the required system reserve ratios $R_{u\ req}$, $R_{f\ req}$, and $R_{d\ req}$, which are given in Table 2 as 1.30, 1.1, and 0.45, respectively. The redundancy ratios are obtained as $r_u = R_u/R_{u\ req} = 1.0$, $r_f = R_f/R_{f\ req} = 1.14$, and $r_d = R_d/R_{d\ req} = 0.82$. Because one redundancy ratio is less than 1.0, this bridge's configuration is considered to be nonredundant.

A bridge system that does not satisfy the redundancy criteria may still be adequate for truck traffic if its members are conservatively designed. Therefore, the redundancy ratios should be combined with a check of member capacity to verify whether overall system safety is still adequate. Checking the member capacity can be performed according to any currently acceptable AASHTO criteria, including WSD, LFD, or the proposed LRFD methods. The next

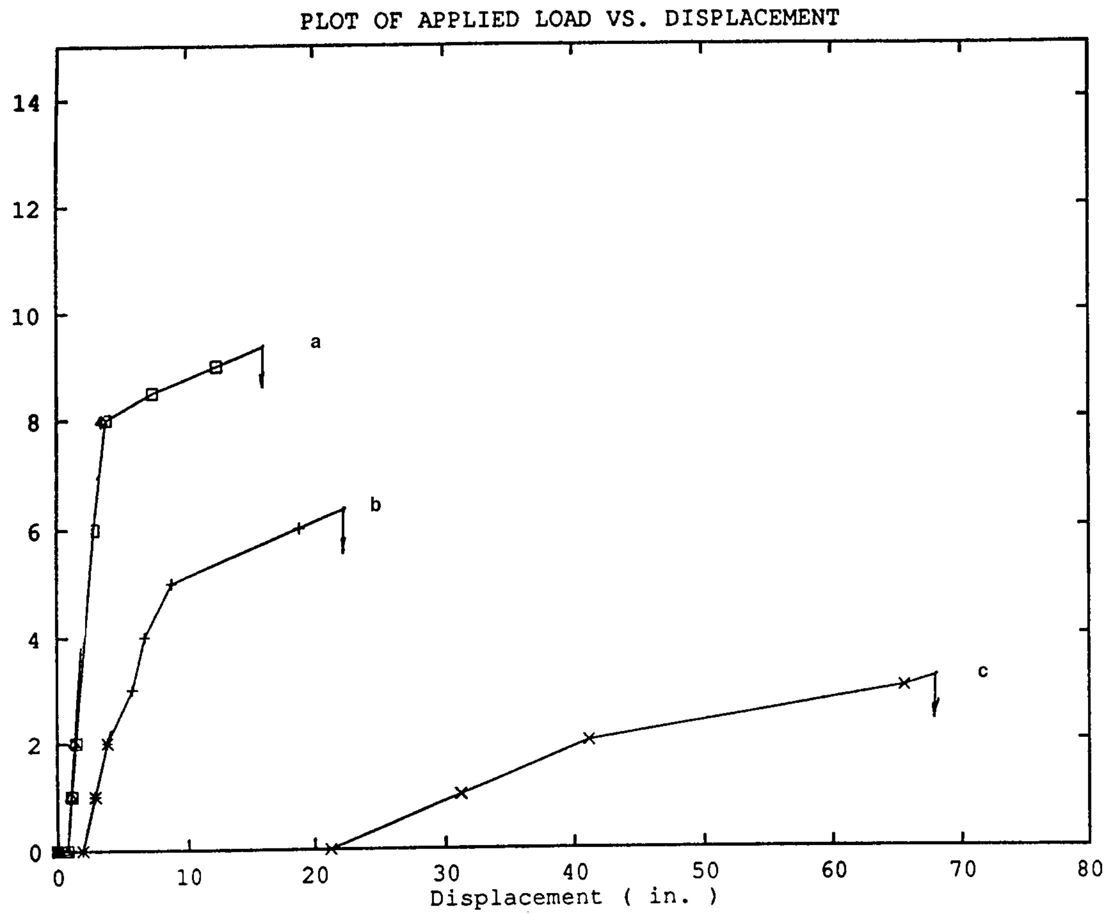


Figure 6. Load factor versus maximum displacement for steel truss bridge.

paragraph illustrates how member safety is checked using the WSD method.

The required member capacity according to AASHTO's WSD criteria is used as the basis of these calculations. If this bridge is to satisfy the proposed WSD criteria, the required capacity of a steel member in tension should be

$$0.55\, R_{req} = D + (1 + I)\, L_{HS\text{-}20} \tag{32}$$

Using the results of the linear elastic analysis with two HS-20 trucks placed on the intact structure as shown in Figure 5, member 29 was found to be the most critical member. For member 29, the dead load effect D is 124 kips. I is the impact factor and, according to the AASHTO specifications, is given as 0.22. $L_{HS\text{-}20}$ is the effect of the AASHTO trucks on the most heavily loaded member and is equal to 67.7 kips. Equation 32 yields a required member capacity R_{req} equal to 376 kips.

The load factor for first member failure required to satisfy the WSD criteria is

$$LF_{1\,req} = \frac{R_{req} - D}{L_{HS\text{-}20}} \tag{33}$$

Substituting R_{req} = 376 kips, D = 124 kips, and $L_{HS\text{-}20}$ = 67.7 kips produces an $LF_{1\,req}$ equal to 3.72. The actual load factor LF_1 is equal to 7.21; this gives a member reserve ratio $r_1 = LF_1 / LF_{1\,req}$ equal to 1.94. This means that member 29 is overdesigned by 94 percent.

The redundancy factor ϕ_{red} is calculated using Equation 18 as

$$\phi_{red} = \min(1.94 \times 1.0,\ 1.94 \times 1.14,\ 1.94 \times 0.82)$$

$$\phi_{red} = \min(1.94,\ 2.21,\ 1.59) = 1.59 \tag{34}$$

This indicates that, according to the WSD criteria, the member reserve capacity (R − D) of all the members of this bridge system can still be reduced by a factor of 1.59 without jeopardizing overall system safety. Thus, although this bridge geometry does not satisfy redundancy criteria, the conservativeness of the member design ensured that enough system safety is still available. For example, the overall capacity of this system would permit this truss bridge to carry sufficient load even when member 12 is assumed to be damaged.

The calculations performed should be repeated for different loading patterns and different damage scenarios. The final ϕ_{red} that must be used is the lowest for all the possible loadings and damage scenarios. In principle, the final ϕ_{red} should be applicable to all the bridge's members. In practice, some bridge members will contribute more than others to overall system redundancy and system safety. Therefore, one may choose to change the strength of one critical member (or a few members) and then repeat the calculations described above to ensure that the system will maintain the required safety criteria.

3.3 CONCLUSIONS

This chapter presented two examples to illustrate the methods proposed in this research project to evaluate the system safety and redundancy of highway bridges. The system factor tables and the incremental analysis approach were used. Despite the different approaches and the different calibrations that were carried out during the development of the proposed techniques, the results obtained from the different approaches are reasonably close. As demonstrated, the proposed methods are practical and relatively easy to use.

CHAPTER 4

CONCLUSIONS AND FUTURE RESEARCH

4.1 CONCLUSIONS

The objective of this study was to develop a framework for considering redundancy in the design and load capacity evaluation of highway bridge superstructures. This goal was achieved by proposing a method to penalize designs with insufficient redundancy by requiring that their members be more conservatively designed than allowed by current standards. On the other hand, adequately redundant designs are rewarded by permitting less conservative member designs. This could be achieved by applying system factors during the routine bridge design and evaluation process. As a first step in the implementation process, and until further experience is gained with the proposed approach, it is suggested that only system factors less than 1.0 be used for new designs while the whole range of factors be used for evaluating existing bridges.

Tables of system factors have been developed for typical bridge configurations. For bridges with configurations that are not covered by the tables, a direct analysis approach is recommended. This requires the performance of an incremental structural analysis to verify whether acceptable behavior, nonfunctionality, or collapse states occur under maximum expected truck loading. Guidelines necessary to perform such an analysis are provided. These include the limit states that should be checked for both intact and damaged conditions as well as the loads that the bridge should sustain before these limit states are violated. Redundancy factors are then calculated from the results of the incremental analysis.

The proposed methods are calibrated using reliability techniques. Redundancy is defined as the difference between the reliability index (or safety index) of the bridge system and the reliability index of the members. The system factor tables and the load factors recommended for the direct analysis approach are calibrated to ensure that highway bridges will provide adequate levels of system capacity.

Although reliability techniques are used during the development of the methodology, the reliability model is transparent to the end user. To consider redundancy during the design and evaluation of a bridge structure, the bridge engineer can simply use the proposed system factors without referring to reliability theory.

The framework proposed in this study to include system redundancy in the bridge design and evaluation process is summarized in a proposed set of specifications that could be included in future editions of the AASHTO *Standard Specifications for Highway Bridges*. The proposed specifications outline how the system factor tables and the direct incremental analysis could be used during the design of new bridges or the load rating of existing bridges.

The calibration process investigated the performance of typical prestressed concrete I-girder and box-girder bridges and multigirder steel bridges. A parametric analysis verified that the redundancy of these bridges is a function of the geometric configuration and is not very sensitive to variations in the section properties. The tabulated system factors were calculated for these bridge configurations assuming that all the members of a bridge are identical. System factors for other configurations can be easily included in the future.

The analysis showed that continuous bridges produced higher redundancy levels than simple-span bridges if the sections in negative bending have sufficient levels of ductility. This means that steel sections in negative bending should be compact and concrete sections should sustain sufficiently high levels of plastic hinge rotations. On the other hand, if the sections in negative bending are not ductile, continuous bridges may show very low levels of redundancy.

4.2 FUTURE RESEARCH

As seen in the examples given in Chapter 3, the system factors provide a simple and accurate tool to include redundancy during the bridge design and evaluation process. The system factor tables provided in this project, however, cover a limited number of bridge configurations. The approach used could be expanded to cover more types of bridges.

Even if more bridge types are included in the system factor tables, some bridges will still require individualized analyses because of their unique configurations. Therefore, attention should also be given to the application of the direct analysis method in practice. The direct analysis method requires the performance of a nonlinear incremental finite element analysis. General purpose finite element packages that can handle nonlinear bridge behavior are commercially available. However, these commercial packages are difficult to use and generally require specialized training. Therefore specialized programs that can be used to analyze bridge structures should be developed and made available to the engineering community. These programs could be modeled after the program NONBAN, which was developed and used in this study to perform the calibration of the system factor tables and the direct analysis method. NONBAN was found to provide accurate results for simple-span as well as continuous bridges for both steel and concrete materials. The program uses a grillage analysis, and the input data requirements are relatively easy to obtain as long as the bridge members can be modeled as one-dimensional beam elements. The program can also be easily expanded in the future to include the nonlinear behavior of complete bridge systems including superstructures as well as substructures.

Chapter 3 provided two examples illustrating the use of the framework developed in this study. More examples should be provided in the future to verify the applicability of the proposed methods for widespread implementation.

During this study it was found that very limited data were available on the behavior of bridge systems at high loads. To execute the reliability calibration, the research team developed a database by performing nonlinear analyses of numerous bridge configurations and different material properties. The calculations used empirical and semi-empirical models on the behavior of steel and concrete bridge members. Although these models are associated with high levels of uncertainty, a number of comparisons have verified that the models used were reasonably accurate. In addition, previous studies have shown that reliability-based calibrations are not sensitive to the database used if the final framework is calibrated to match the performance of existing adequate designs. More sensitivity analyses could be performed in the future to further verify the robustness of the proposed framework.

The analyses in this study focused on the behavior of the main bridge members. Although the nonlinear behavior of the deck was included by modeling it as several parallel beam elements, the failure of the slab due to crushing of the concrete under transverse bending or due to shear failure was not considered. Several experimental analyses have verified that failure of the decks was not observed during full-scale field tests. This observation is, however, conditional on having a properly designed slab. Therefore, possible failure of the deck should be included in the future.

This study was concerned with the redundancy of highway bridge superstructures. The general framework is, however, applicable to the substructure as well, as in Appendix E. Because many bridge failures occur due to failure of substructure elements, future studies should expand the models developed here to consider the redundancy of complete bridge systems including the substructure as well as the superstructure.

The load models used in this study were adapted from the models developed for the LRFD code calibration studies. These load models were developed on the basis of the linear behavior of bridge systems and using a statistical database on truck traffic and truck gross weights obtained throughout the United States and Canada. Many bridge sites, however, have unique permit load and posting requirements that would make the general database used for the calibration of design codes unsuitable for use when performing the evaluation of existing bridges. For this reason, the extension of the framework proposed in this project to include site-specific load data should be undertaken in future studies.

REFERENCES

1. *Standard Specifications for Highway Bridges.* AASHTO, Washington, DC (1996).
2. *AASHTO LRFD Bridge Design Specifications.* AASHTO, Washington, DC (1994).
3. Nowak, A., "Calibration Report for NCHRP Project 12–33." Department of Civil Engineering, University of Michigan, Ann Arbor, MI (May 1992).
4. Galambos, T. V., Leon T. R., and French, C.W., *NCHRP Report 352,* "Inelastic Rating Procedures for Steel Beam and Girder Bridges." National Research Council, TRB, Washington, DC (1992).
5. Moses, F. and Ghosn, M., "A Comprehensive Study of Bridge Loads and Reliability." Report FHWA/ODOT/85-005, U.S. Department of Transportation, Washington, DC (1985).
6. Throft-Christensen, P. and Baker, M. J., *Structural Reliability Theory and Its Implications.* Springer-Verlag, New York (1982).
7. *Ontario Highway Bridge Design Code.* Ontario Ministry of Transportation and Communications, Toronto, Ontario, Canada (1983).
8. Moses, F. and Verma, D., *NCHRP Report 301,* "Load Capacity Evaluation of Existing Bridges." National Research Council, TRB, Washington, DC (December 1987).

APPENDIXES A THROUGH F

UNPUBLISHED MATERIAL

Appendixes A through F contained in the research agency's final report are not published herein. For a limited time, loan copies are available on request to NCHRP, Transportation Research Board, Box 289, Washington, DC 20055. The appendixes are titled as follows:

Appendix A: Literature Review
Appendix B: Reliability and Redundancy of Prestressed Concrete I-Beam Bridges
Appendix C: Reliability and Redundancy of Steel I-Beam Bridges
Appendix D: Reliability and Redundancy of Prestressed Concrete Box-Girder Bridges
Appendix E: Redundancy in Bridge Substructures
Appendix F: Documentation of the Program NONBAN